CODE OF PRACTICE
FOR
LIMIT STATE DESIGN OF FOUNDATIONS

INDIAN ROADS CONGRESS
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CODE OF PRACTICE ON LIMIT STATE DESIGN FOR FOUNDATIONS

INTRODUCTION

The present IRC:78, Standard Specifications and Code of Practice for Road Bridges Section VII – Foundation and Sub-Structure is based on working stress design. The Indian Roads Congress had taken a decision to adopt Limit State Design approach for design of all components of bridge structures and had directed various Committees to evolve codes on Limit State Approach. The work of Foundation design, based on Limit State was assigned to B-3 Committee. The document was drafted by Mr. T. Viswanathan. It was discussed in several meetings of the B-3 Committee and was revised a number of times. The Committee also decided that an Explanatory Note and Worked out examples, using both the limit state approach and working stress approach be also included in the document for easy understanding of the users. The worked-out examples were provided by Mr Navneet Gupta and Mr Sanjay Kumar Jain.

The draft document was approved by B-3 Committee in its 11th meeting held on 2.8.2019. Since revision of IRC:78 is likely to take some more time the Committee decided to recommend publication of this document as IRC:78-Part 2. The B-3 Committee also recommended that option may be given to designers for the next 2 years to follow either the working stress method as per present IRC:78 which will be renumbered as IRC:78 Part 1 on revision or as per Limit State approach given in this Part 2 for design of foundations for bridges.

The Draft Document was approved by the Bridges Specification and Standards Committee in its meeting held on 6.12.2019 with certain observations. The draft was modified as per observations of BSS Committee. Subsequently, the draft was approved by the Executive Committee on 19.12.2019. The draft was discussed and approved by the Council of the Indian Roads Congress in its 219th meeting held at Patna on 19.12.2019.

The designer may carry out deign of foundation using the current provision as per IRC:78 using working stress philosophy with unfactored loads or as per this code using partial load factors and material resistance factors. Both these philosophies will continue to operate simultaneously for such period till provision of working stress method is withdrawn.

References have been made to certain clauses of other IRC and IS codes. Those are as prevailing in December, 2019. Any revision in those codes may be duly considered by the designer.

The task was completed by B-3 Committee with the following personnel:

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1. SYMBOLS

1) Latin Letters

A_c	The area of base of foundation in contact with soil
L, B	Dimensions of foundations
CR1 CR2 CR3 CR4 CR5 CR6	Correlation factors
C_u	Un-drained shear strength
C_{ud}	Design value of un-drained shear strength
d	Diameter of circular foundation
e	Eccentricity of vertical loads in longitudinal direction for single axis eccentricity
e_B	Eccentricity of vertical loads in transverse direction
e_L	Eccentricity of vertical loads in longitudinal direction
GR1 GR2 GR3 GR4 GR5 GR6 GRS GRA	Partial factor for ground Resistance in case of pile foundations
R1, R2	Partial factor for ground resistance in case of open foundation
H_d	Factored design horizontal force at base foundation
M	Model factor
Q_{Rd}	Reduced capacity of pile based on partial factor for ground Resistance
Q_{td}	Design shaft tensile resistance of pile
Q_{tk}	Characteristic shaft tensile resistance of pile
Q_{uc}	Characteristic compressive resistance or capacity of pile
Q_{ubc}	Characteristic base resistance of pile
Q_{ufc}	Characteristic shaft resistance of pile
Q_{ud}	Ultimate design resistance of pile

R_d	Design value of resistance
V	Total design vertical load acting normal to foundation base

2) Greek letters

Δ	Structure ground interface friction angle (wall or base interface)
γ_b	Partial factor for base resistance of pile
γ_c	Partial factor for effective cohesion
γ_{cu}	partial factor for un-drained shear strength
γ_m	Partial factor for soil parameter
γ_{qu}	Partial factor for unconfined compressive strength
γ_s	Partial factor for shaft resistance of a pile
γ_{st}	Partial factor for tensile resistance of pile
γ_t	Partial factor for combined resistance of pile or Partial factor for the density of the soil
ϕ	Angle of shearing resistance of soil
ϕ'	Design angle of shearing resistance of soil.
σ_v	Base pressure

For terms and definitions Section 3 of IRC:112 may be referred.

ULS	Ultimate Limit State
SLS	Serviceability Limit State
UCS	Ultimate Compressive Strength
SV	Special Vehicle
FOS	Factor of Safety

2. DESIGN PHILOSOPHY AND VERIFICATIONS UNDER ULTIMATE AND SERVICEABILITY LIMIT STATES

2.1 Design Philosophy

Limit state design philosophy will ensure that the foundation will be safe with regards to the various limit state requirements under different design situations. The philosophy will ensure that the limits stated herein are not exceeded during the life of the structure.

The limit states can be divided into two types, Ultimate Limit State (ULS) and Serviceability Limit State (SLS).

Under the ultimate limit state, the loss of equilibrium of the structure, excessive deformation of the ground (no bearing failure) and loss of equilibrium of the structure or the ground due to uplift by buoyancy, and adequacy of capacities of various elements etc. are verified. For illustration the modes of such failures of open foundation are shown in **Fig. 1** and **2**.

Under serviceability limit state, the contact area of base, the bearing pressure under open and well foundations and settlement for the foundations are to be investigated.

The procedure outlined in the following sections shall be used for, assessing the effects on structure and elements resulting from the geotechnical actions which shall be appropriately combined with effects of other relevant actions to design the foundations. The structural design of members of foundations shall be carried out in accordance with IRC: 112 and IRC:24 for members in concrete and steel respectively. For substructures subjected to geotechnical force (e.g., abutment, retaining wall excluding reinforced earth wall, abutment well cap and pile cap) the structural design shall be carried out according to relevant codes for load combinations given in **Table B.4** of IRC: 6 for ultimate limit state using appropriate material safety factors on soil and as per **Table B.3** of IRC: 6 for serviceability limit state.

The Code is applicable for geotechnical design of open foundation, well (Cassion) foundation, and pile foundation with limit state approach.

2.2 Combinations of Loads and Forces for Limit State Design of Open Foundation

The verifications shall to be carried out under the ultimate limit state for loss of equilibrium and bearing resistance failure for combinations of loads and forces as per Clause 2.2.1. The verification under serviceability limit state shall be carried out as per Clause 2.2.2.

2.2.1 *Verification under ultimate limit state*

Verifications shall be carried out for:

a) Loss of equilibrium (i.e. overturning),
b) Overall stability,
c) Sliding of foundation and uplift,
d) Bearing resistance failure, and
e) Loss of equilibrium due to uplift of structure subjected to hydraulic uplift.

The verifications for a) to c) and e) shall be carried out using loads and forces as per **Table B.1** of IRC:6 and partial material safety factor as per **Table 1**.

Bearing resistance failure shall be verified using the following combinations of loads and forces given in **Table B.4** of IRC:6, and partial factor on materials and ground resistance factors as shown under relevant clauses of this code:

i) Load combination 1 + partial factor on materials set 1 value+ resistance factor R1.
ii) Load combination 2 + partial factor on materials set 2 value + resistance factor R1
iii) Seismic combination + partial factor on materials set 2 value + resistance factor R2
iv) Accidental combination + partial factor on materials set 1 value + resistantce factor R1

(Meaning of "+" is combined with)

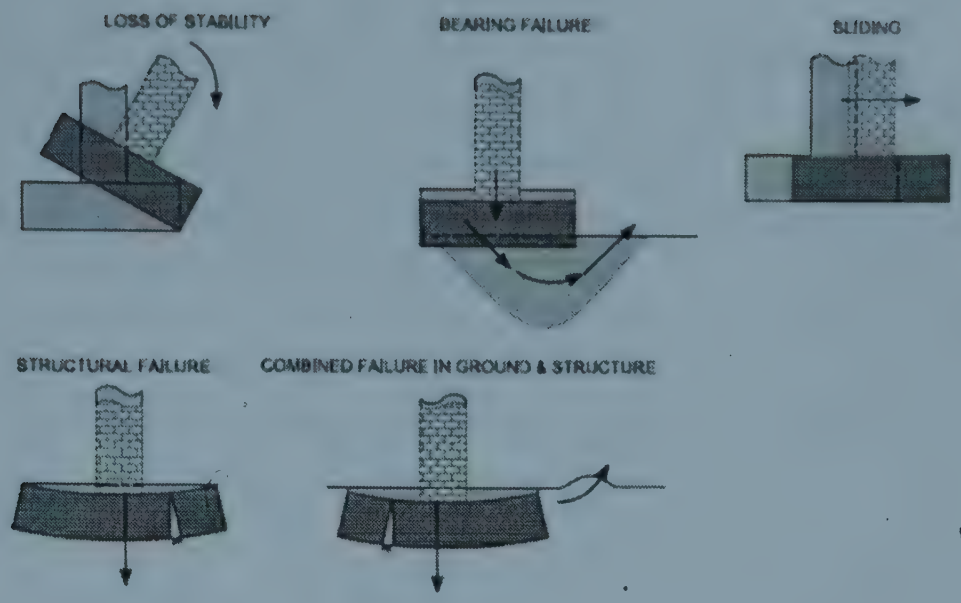


Fig. 1 Failure of Shallow Foundations

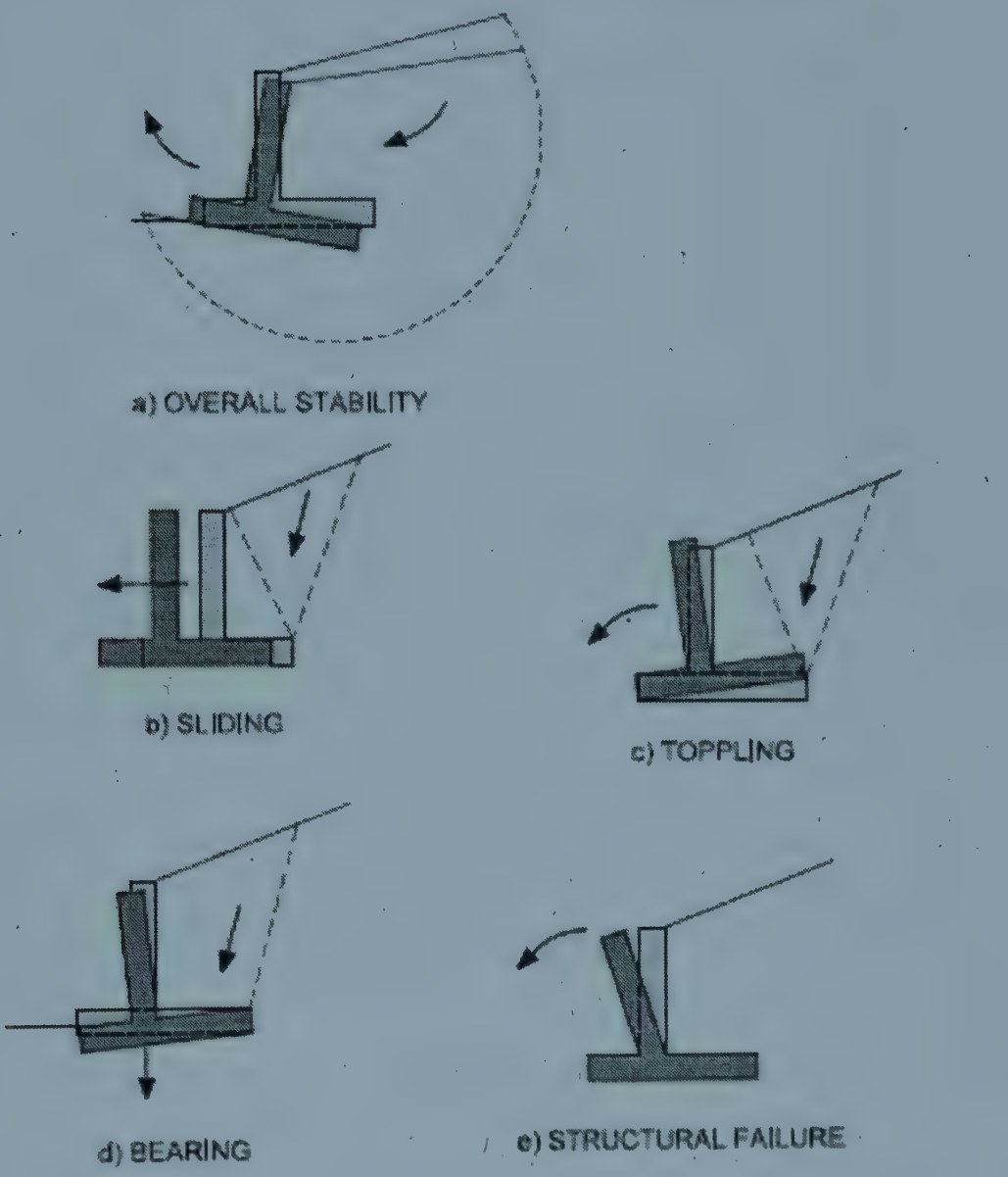


Fig. 2 Failure of Reinforced Concrete retaining wall

2.2.2 *Verification under serviceability limit state*

Under serviceability limit state the following verifications shall be carried out

(a) Contact area of base where applicable under frequent combination

(b) Base pressure under rare combination

(c) settlement under quasi permanent combination

3. OPEN FOUNDATION

3.1 Check for loss of equilibrium:

The structure loses its equilibrium when it overturns or slides or gets uplifted. Hence it shall be ensured that the structure neither over turns nor slides. The uplift verification shall necessarily be carried out where buoyancy forces are significant.

3.1.1 *Verifications against overturning and overall stability*

Verification against overturning for all structures and over turning and overall stability for foundations supporting the retaining structure or abutments shall be carried out using the partial factors for loads and forces given in **Table B.1** of IRC: 6 for all the combinations. The partial factor for water pressure shall be taken as 1.0, if encountered. However partial factor needs to be applied on the geo technical materials which are which are given in **Table 1**, while computing the effect of geo-technical actions for converting characteristic values to design values.. The computed geotechnical forces (Loads) are to be further multiplied by partial factor given in **Table B.1** of IRC:6. The design parameter for geo technical material to be adopted for computation of geo technical force shall be calculated from the characteristic parameter. Design Parameter = Characteristic Parameter obtained from tests/γ_m except for design angle of shearing resistance. The design angle of shearing resistance shall be computed as per note given below **Table 1**.The partial factor on materials shown in **Table 1** are applicable to check stability (e.g. over turning, over all stability, sliding and uplift).

Table 1 Partial Factor for Soil Parameter (γ_m) for stability verification

Soil Parameter	Symbol	Partial Factors
Angle of Shearing Resistance	ϕ	1.1
Effective Cohesion	γ_c	1.1
undrained Shear Strength	γ_{cu}	1.2
Unconfined compressive strength	γ_{qu}	1.2
Density	γ_t	0.9

Note: Design angle of shearing resistance = $\phi^1 = \tan^{-1}(\tan\phi/\gamma_m)$

It shall be ensured that the overturning moment ≤ Resisting Moment

3.1.2 *Verification against sliding*

The verification against sliding shall be carried out by using the, partial factors for loads and forces as per **Table B.1** of IRC:6. The partial factor for materials for computing the geo technical forces shall be adopted given in **Table 1**. It shall be ensured that factored design horizontal forces are less than the resisting forces.

i) Verification for foundations, except on cohesive stratum in undrained condition:

$H_d \leq V\tan\delta$ **Eq. 1**

Where,

H_d = Factored design horizontal forces acting at base of foundation

V = The factored design vertical forces acting at base of foundation

$\tan\delta$ = can be taken as $\tan\phi^1$ between soil and concrete, 0.95 between good rock and concrete, and 0.85 between fissured rock and concrete.

ii) Verification for foundations on cohesive stratum in undrained condition:

$$H_d \leq R_d \qquad \textbf{Eq. 2}$$

Where,

R_d = The design resistance (R_d) for cohesive soil in undrained condition should be taken as

$$R_d = A_c\, C_{ud}$$

Where,

A_c = The area of base in contact as per Clause 3.2.3

C_{ud} = The factored undrained design shear strength = C_u / γ_{cu}.

This verification shall be carried out after calculating the contact area of the base as per Clause 3.2.3.

3.2 Verification for Bearing Resistance

The pressure transmitted by the foundation shall not exceed the ultimate bearing resistance under any combination.

3.2.1 *Partial factor for loads and combination of loads*

The bearing resistance shall be verified for the factored loads and forces and their combinations Table given in **Table B.4** of IRC: 6. Wherever water pressure is encountered, the partial factor shall be taken as 1.0 for the same.

3.2.2 *Partial factor on materials*

Geotechnical forces, bearing capacity or ground capacity for design of foundations shall be computed using the design properties for the materials which are obtained from characteristic properties by using the partial factors for materials given in **Table 2**. The method of obtaining the design parameters for geotechnical materials from the characteristic parameters shall be as specified in Clause 3.1.1.

Table 2 Partial Factor for Soil Parameters (γ_m) for computing the geotechnical forces and bearing capacity

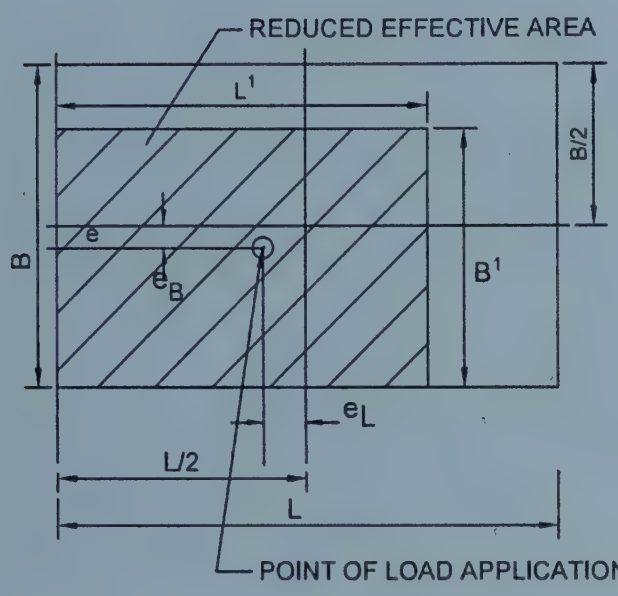
Soil Parameter	Symbol	Set 1	Set 2
		To be used in Combination (1) and in Accidental Combination	To be used in Combination (2) and in seismic combination
Angle of Shearing Resistance	ϕ	1.0	1.25
Effective Cohesion	γ_c	1.0	1.25
Undrained Shear Strength	γ_{cu}	1.0	1.4

unconfined compressive strength	γ_{qu}	1.0	1.4
Density	γ_t	1.0	1.0

Note: Design angle of shearing resistance = $\phi^1 = \tan^{-1}(\tan\phi/\gamma_m)$

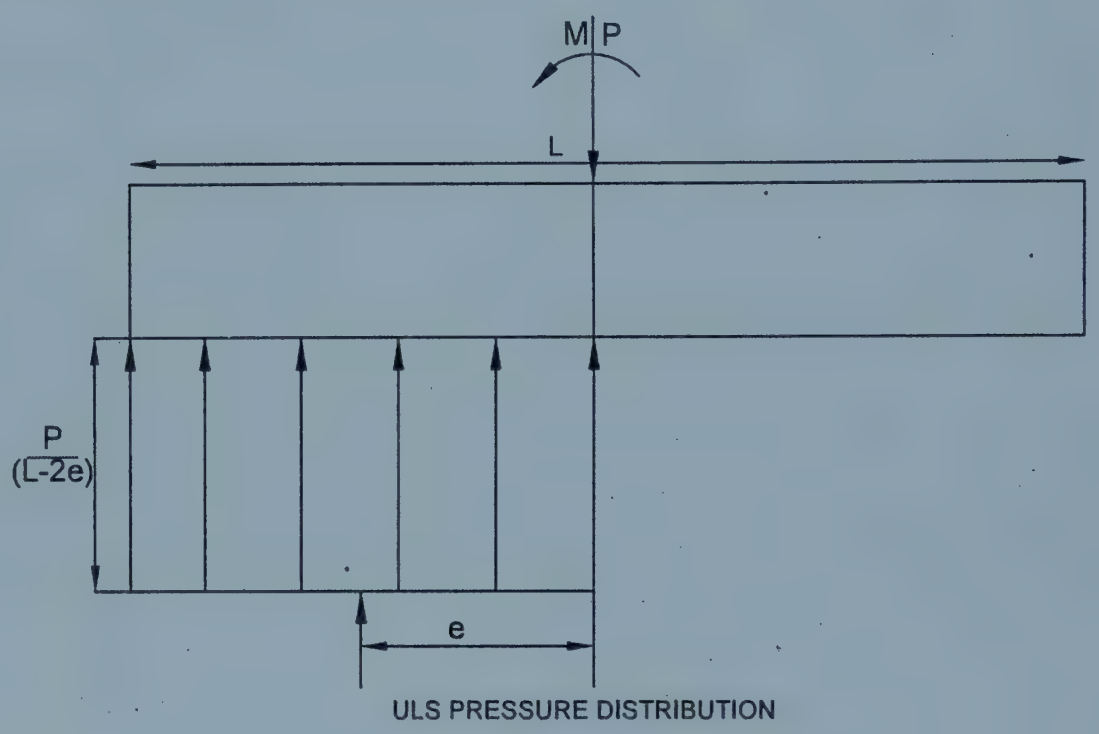
3.2.3 *Calculation of bearing pressure for foundations resting on Soil:*

When foundation is subjected to moment about one or both axes, the base pressure shall be calculated assuming a uniformly distributed pressure over an effective base area. The centre of gravity of total vertical loads, including self load of the base and with due consideration to eccentricity of loads, shall coincide with the center of gravity of the effective base area. The pressure on effective area shall be assumed to be uniform, equal to total vertical load divided by effective base area. The principle is illustrated for rectangular footing in **Fig. 3** and **4.**



Reduced Footing Dimensions

Fig. 3 Double Axes Eccentricity



ULS Pressure Distribution

Fig. 4 Single Axis Eccentricity

In case of rectangular footing subjected to load with eccentricity about one axis, the bearing pressure transmitted is as below:

$$\sigma_v = \Sigma V/(L-2e)\,(B) \qquad \textbf{Eq. 3}$$

In case of rectangular footing subjected to load with eccentricity about both axes the bearing pressure transmitted is as below:

$$\sigma_v = \Sigma V/(L-2e_L)\,(B-2e_B) \qquad \textbf{Eq. 4}$$

ΣV = Total design vertical load acting normal to foundation base

σ_v = The bearing pressure transmitted.

L and B are dimensions of foundation

L^1 abd B^1 are reduced dimensions of foundation in contact with base stratum

e, e_L and e_B are eccentricities.

In case of circular footing the bearing pressure transmitted will be as below:

$\sigma_v = \Sigma V$/Effective area of contact. **Eq. 5**

Table 3 gives the contact area for typical e/d ratios. For any other ratio the contact shall be calculated.

Table 3 Area of Contact

e/d	0.1	0.125	0.15	0.20
Area of contact	76%	70%	64	50%

It shall be ensured that the minimum contact of base area both for rectangular and circular foundation, shall not be less than 50% of actual area of the foundation under all four combinations.

The bearing pressure as per this clause is a check for safety of the foundation size. For structural design purpose the base pressure shall be calculated as per Clause 3.2.5.

3.2.4 *Calculation of Bearing Pressure on Rock*

When the foundation is supported on Rock, the base pressure shall be calculated by assuming a linearly varying pressure over effective base area. The contact pressure diagram will be either triangle or trapezoidal.

The centre of gravity of load system shall coincide with centre of gravity of effective base area. The minimum contact area of base of 67% shall be ensured for all four combinations. When the Unconfined Compressive Strength (UCS) of the base stratum equal to or more than 4 Mpa, it shall be treated as rock for the purpose of this Clause.

3.2.5 *Calculation of bearing pressure for structural design of foundation*

For the structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact pressure depending upon the eccentricity based on the factored loads shall be worked out and used both for the foundations resting on soil and rock. The structural design of foundation shall be carried out as per IRC:112.

3.2.6 *Calculation of allowable bearing capacity. (Resistance)*

The ultimate design net bearing capacity for soils shall be calculated as per IS: 6403 using the partial factor for materials as per **Table 2** for the respective combinations. The reduced dimensions corresponding to effective area of the foundation shall be used for estimating the ultimate net bearing capacity which will lead to calculation of bearing capacity for each combination. The bearing capacity thus worked out shall be further divided by the following ground resistance factors to arrive at the allowable ultimate design bearing capacity.

Resistance factor R1 for foundations resting on soil shall be 1.3 for all combinations except for seismic combination. For seismic combination resistance factor R2 shall be 1.1. For foundations resting on rock, the resistance factor shall be 4.5.

3.3 Verification of Uplift Limit State

Verification of uplift limit state is required for structures subjected to uplift force e.g. an underpass subjected to uplift due to ground water level raising above the foundation level. The check shall

be carried out for the combinations shown in **Table B.1** of IRC: 6. The partial factor for loads shall be adopted as per Table mentioned above. Any additional resistance (R_d) to counter the uplift is mobilized either by providing tension piles (shall be considered only for resisting uplift force) or anchors or both. The design additional resistance shall be computed by dividing the calculated characteristic additional resistances (R_d) by the partial factors given below:

For Anchors 1.1

For tensile pile resistance as given under pile foundation section (**Table 10**). For soil parameters to be used in the computation of the resistance, the partial factors on materials shown in **Table 1** shall be applied (Angle of Shearing resistance, effective Cohesion, undrained Shear Strength, Density). Verification for uplift shall be carried out by checking the design value of combination of unfavorable uplift forces is less than or equal to sum of resisting forces and of any additional resistance to uplift. V is the vertical loads or forces.

V unfavorable < V favorable + Design additional Resistance

Verification against uplift failure of the block of ground containing the piles and ground anchors shall also be carried out using partial factors on materials shown in **Table 1**.

While considering the uplift, of the block of ground containing the tension piles, the shear resistance along the sides of the block, the self weight of soil in the block and weight of piles shall be added to other resisting self weights if any with the partial factor shown in **Table B.1** of IRC: 6 for stabilizing actions

Such foundations where tension piles are provided to resist the uplift shall not be treated as piled raft foundation or pile foundations for resisting compressive loads.

3.4 Verification of Serviceability Limit State

Verification of serviceability limit consists of verification contact area of base, bearing pressure and settlement under the foundation. For these verifications the partial factor on materials shall be taken as 1.0

3.4.1 *Verification of contact area of base and bearing pressure*

The contact area of base under foundation shall be verified using the frequent Combination of loads shown in **Table B.3** of IRC:6. The partial factor for buoyancy shall be taken as 1.0

For foundation resting on soil, at least 90% contact area of the base shall be ensured. In case of foundations resting on rock, at least 80% contact area of base shall be ensured. The pressure distribution under foundation shall be worked out based on either trapezoidal on triangular variation. The maximum base pressure shall be checked under the rare combination of action as per **Table B.3** of IRC:6.

The factor of safety for working out allowable base pressure from ultimate bearing capacity shall be as below:

For foundations resting on soil 2.25

For foundations resting on rock 7

3.4.2 *Calculation of Settlement*

For the calculation of settlement under the foundation the Quasi permanent combination as per **Table B.3** of IRC:6 shall be used. The settlement calculation shall be as per IS:8009 Part 1.

Rapid settlement may occur in cohesion less or granular soils (medium-dense or firm layer overlaying rock or hard soil) often as the foundation is loaded. Settlement occurring during the construction of foundation and substructure is not critical for serviceability and functioning of the bridge. Foundations resting on this kind of soil, calculation of settlement shall be carried out using superstructure loads only. However for computation of settlement for foundation resting on cohesive soil, the immediate settlement shall be computed using superstructure loads and for computation of total settlement including consolidation settlement, total loads shall be used. The total settlement shall comprise of both immediate and long-term settlements.

3.4.3 *Differential settlement*

The differential settlement shall be checked between adjacent foundations. The angular distortion of superstructure resulting on account of differential settlement shall not exceed 1.in 400.

3.5 Special Vehicle (SV) Loading

3.5.1 *Verifications*

Verification of equilibrium, base pressure and strength checks shall be carried out under ULS for load combinations 1 and 2 as per Clause 2.2.1.

Verification of base pressure and strength check for rare combination of loads under SLS shall also be carried out. The Partial factor on SV loading shall be taken as 1.15 as per in Clause 204.5.4 of IRC:6 for ULS verifications except for combination 2. For combination 2 and rare combination under SLS, the partial factor on SV loading shall be taken as 1.0.

3.5.2 *Method of Verification*

For the load combinations all the relevant clauses for verification with loads and combinations, already specified in this Code shall be applicable with the following modifications for carrying out the verifications for this load.

(a) The ground resistance factor shall be 1.2 as against 1.3 mentioned in Clause 3.2.6 for arriving at the allowable ultimate bearing capacity under combination 1 and 2 for foundations resting on soil. For foundations resting on rock the ground resistance factor shall be taken as 4.5

(b) The factor of safety of 1.75 shall be ensured on ultimate bearing capacity as against 2.25 mentioned in Clause 3.4.1 while checking the base pressure under rare combination of loads for foundations resting on soil.

(c) Factor of safety of 6.0 shall be ensured as against 7.0 mentioned in Clause 3.4.1 while checking the base pressure under rare combination of loads for foundations resting on rock.

3.6 Other provisions

Other provisions for open foundation not covered in the above clauses shall be governed by the provisions given in Clause 705 and 707 of IRC: 78.

4. PILE FOUNDATION

4.1 General

This section covers the design of end-bearing pile, friction pile, tension pile and laterally loaded pile installed by boring or driving, resting on Soil or Rock or Intermediate Geo-materials. The sub sections cover Actions, Combination of Actions, estimation of axial capacity of pile, acceptance criteria, design of pile subjected to axial loads and/or lateral loads.

4.2 Loads and Load Combinations

The loads to be considered for the design are given in **Table B.4** of IRC:6 along with their partial factors for actions for various combinations. Combinations, to be considered are load combination 1, load combination 2, seismic combination and accidental combination. Certain actions are not covered in the above Table and the same are indicated below along with their partial factors.

(a) Negative skin friction effect from the ground. The partial factor for this force, in combination 1 and in accidental combination shall be taken as 1.35 and for the remaining combinations it shall be 1.0.

(b) Pile foundations are subjected to lateral Geo-technical actions such as, due to different amounts of surcharge on either side of foundation (e.g near abutment), due to different level of excavation on either side, foundation constructed on slope etc. The partial factor for this loading in combination 1 shall be 1.50 and in Combination 2 shall be1.3 and in other combinations it shall be 1.0.

(c) Pile foundations in seismic zone are also be subjected to lateral loads due to seismic action. The partial factors on sesismic actions shall be as specified in IRC 6.

4.3 Set value of partial factor (γ_m) for Geo-technical materials to be used for computation of force and ground Resistance

The Set to be used for computation of geotechnical force and ground resistance from Geo-technical materials (soil, rock and intermediate Geo-materials) under different load combinations is given in **Table 4**. The partial factors for the relevant Set may be obtained from **Table 2**.

Table 4 Set Values of Partial Factor(γ_m) to be used for Material for Computation of Geo Technical Force/Ground resistance.

S. No.	Combination	Set
1.	For computation of ground resistances (both in vertical and in lateral direction), negative skin friction effects, and estimation of geotechnical lateral forces including those specified in Clause 4.2 for combination 1	1
2.	For computation of ground resistance in vertical direction in Combination 2, and in seismic combination	1
3.	In Combination 2 and in seismic combination for estimation of ground resistance in lateral direction, computation of negative skin friction effects and estimation of geotechnical lateral forces including those specified in Clause 4.2	2
4.	In accidental combination for estimation of ground resistance in vertical direction and estimation of negative skin friction effects	1
5.	In accidental combination for estimation of ground resistance in lateral direction and estimation of geo technical lateral forces including those specified in Clause 4.2.	2

The vertical ground resistance shall be computed always using set 1 value only for materials, in all combinations for piles subjected to both compressive and tensile forces. The partial factor for ground resistance for combination 1 is GR1 and for combination 2 is GR2, GRS for seismic combination and GRA for accidental combination which shall be obtained from **Table 5**. For computation of negative skin friction effect on the piles, the characteristic undrained shear strength or unconfined compressive strength (for calculation of shear strength,) the material safety factor shown in **Table 2** shall be used as multiplier and not as a divider to arrive at the design value from the characteristic value Taking into account the actions, combinations, partial factor on materials and ground resistance factors, the combinations can be expressed as

Load combination 1 + Partial factor on materials set 1 value + Ground resistance factor GR1

Load combination 2 + Partial factor on materials set 2 or set 1 value as per above table + Ground resistance factor GR2

Seismic combination + Partial factor on materials set 2 or set 1 value as per above Table + Ground resistance factor GRS

Accidental combination + Partial factor on materials set 2 or set 1 value as per above Table + Ground resistance factor GRA

(Meaning of "+" is combined with.)

4.4 Ultimate Limit States Verifications

The following ultimate limit states shall be verified for the pile foundation.

(i) Loss of Equilibrium
(ii) Bearing Resistance Failure of the pile foundation through Pile Capacity
(iii) Uplift or tensile resistance of pile
(iv) Failure of the ground due to lateral loading on pile
(v) Structural failure of pile

4.4.1 *Loss of Equilibrium*

Loss of equilibrium, if required needs to be checked for short pile foundations only as the long pile foundation is not liable to lose equilibrium.

4.4.2 *Verification of bearing resistance for piles resting on soil through pile capacity.*

4.4.2.1 Estimation of design axial compression load on a pile and pile groups.

The design axial load on a pile shall be estimated for all the 4 combinations described in clauses 4.2 and.4.3. The design axial compression load on a pile or on a group of piles shall not exceed the corresponding axial capacity based on bearing resistance of the ground to avoid bearing resistance failure. The self weight of pile with associated partial factor should be included while calculating the design axial compression load under the following circumstances.

1) The down ward drag is significant
2) The exposed pile length of pile above scour level is large (> 60% of total length)

When the above conditions are not met with, then the common practice of assuming that the weight of the pile is balanced by that of over burden allowing both to be excluded from load as well as from resistance can be followed.

4.5 Calculation of Pile Capacity of Piles Resting on Soil

The pile capacity may be calculated by any one of the following methods.

a) By calculation, using the ground parameters obtained from site investigation
b) By calculation, from profiles of ground test results obtained from in-situ site investigation
c) Directly from static pile load test
d) From Dynamic pile load tests
e) From pile driving formula

For the methods mentioned in a), b), d) and e) the static pile load test is required to be conducted as specified in Clause 4.5.3 adhering to all requirements for verifying the theoretical capacity worked out. For river bridges the scour effect shall be given due consideration while calculating the capacity of pile and also during load testing.

4.5.1 *Calculation of pile capacity using ground parameters from site investigation*

The ground parameters can be obtained either from field in-situ tests such as vane shear test, SPT or from lab tests on samples taken from the site. This method shall be adopted when the capacity is calculated using equation or charts

Characteristic pile resistance (pile capacity) of the pile Q_{uc} is determined by calculating the end bearing resistance and skin friction resistance along the pile surface based on the soil parameters. The method of calculation of characteristic pile resistance or ultimate pile resistance (Q_{uc}) is given in **Appendix – 5** of IRC:78. The characteristic capacity thus arrived shall be further divided by respective partial factor for ground resistance GR as per **Table 5** and model factor M in order to arrive at the design pile resistance (pile capacity) Partial factor for ground resistance will also depend upon the method of installing the piles. If the pile resistance is calculated at base and shaft components separately, then these resistances are to be divided by respective

partial ground resistance factors to obtain reduced resistances (capacity). The partial factor for ground resistance shall be as per **Table 5**. For calculating reduced pile capacity the following formulae shall be used:

The partial factor for ground resistance shall be as per **Table 5**. For calculating reduced pile capacity the following formulae shall be used:

$$Q_{Rd} = Q_{ubc}/\gamma_b + Q_{ufc}/\gamma_s \qquad \textbf{Eq. 6}$$

Where

Q_{ubc} = Characteristic base resistance

Q_{ufc} = Characteristic shaft resistance

Q_{Rd} = Reduced capacity of pile based on partial factor for ground resistance.

Table 5 Partial Factor for Ground Resistance for Shaft in Compression

Component	Symbol	For bored and continuous auger Pile		Driven Pile		GRS for Seismic Combination	GRA for accidental combination		Model Factor for bored and auger pile and driven piles
		GR1	GR2	GR1	GR2		For Vehicle collision, Log Impact and barge collision	Ship Collision	M
For Base resistance	γ_b	1.0	1.7	1.0	1.5	1.40	1.25	1.40	1.2
For shaft resistance	γ_s	1.0	1.4	1.0	1.3	1.15	1.25	1.15	1.2
Total combined resistance to be used. When shaft and base resistance are not calculated seperately	γ_t	1.0	1.7	1.0	1.5	1.40	1.25	1.40	1.2

Resistance factors shown under GR1 shall be used for arriving at the reduced resistance under load combination1, Resistance factor shown under GR2 shall be used for arriving at the reduced resistance under combination 2. GRS resistance factor shall be used for arriving at the reduced resistance under seismic combination. GRA shall be used for arriving at the reduced resistance under the respective accidental situations. In order to arrive at the design pile resistance or capacity, Q_{Rd} shall be further divided by a model factor M.

$$Q_{ud} = Q_{Rd}/M \qquad \textbf{Eq. 7}$$

Where Q_{ud} = The ultimate design resistance or capacity of pile.

M = The model factor.

It shall be ensured that the design axial load imposed on piles in all combination V_{cd} ≤ Allowable design resistance or capacity of pile Q_{ud}

The characteristic ultimate capacity of pile, (Q_{ubc} + Q_{ufc}) shall be load tested and ensured that the displacement is less than 10% of the pile diameter. The number of tests and correlation factor shall be in accordance with **Table 7**.

4.5.2 *Calculation of Pile capacity from profiles of ground test results obtained from in-situ site investigation*

The static cone penetration test or pressure meter in-situ test shall be conducted at the project location. Complete in-situ profile of the soil shall be obtained in order to adopt this method. Using the observed parameters, the capacity of piles shall be obtained based on the resistances for each location. The procedure as per **Appendix-5** Clause 7 of IRC:78 shall be adopted in case of cone penetration tests. This method is strictly applicable when the investigation is carried out by in-situ method for obtaining the profile of the ground and no equation or charts are used to compute the pile capacity. This is termed as calculated capacity of pile.

The mean calculated pile capacity shall be obtained from the calculated pile capacity for the different locations. The lowest calculated capacity also shall be identified. Depending upon the number of test locations, correlation factor (or the weightage factor) has also been suggested. The correlation factor as per **Table 6** shall be used to arrive at the characteristic capacity of pile from the calculated capacity based on in-situ test results.

Table 6 Correlation Factor for obtaining the Characteristic Capacity of Pile from Ground Profile Test Results

No. of tests	1	2	3	4	≥ 5
CR1	1.40	1.30	1.29	1.25	1.23
CR2	1.40	1.26	1.20	1.17	1.14

The characteristic calculated capacity of pile = Minimum of [Mean calculated capacity/CR1 or Minimum calculated capacity/CR2]. (Minimum of the two to be taken as characteristic pile capacity)

The characteristic calculated capacity thus obtained shall be further divided by the partial ground resistance factors shown in **Table 5** for arriving at the design resistance of pile. The resistance factor shall be applied on total capacity as a single factor or can be applied separately on to the friction and bearing components. If the capacity is arrived using two components the following procedure shall be followed:

i) The characteristic capacity of pile = Minimum of ((mean of base capacity + mean of shaft capacity)/CR1 or (Minimum of (base capacity + shaft capacity)/CR2). For arriving at the minimum capacity the base and shaft resistances have to be taken together for the same location

ii) If the mean capacity is governing, then the design capacity of pile = (mean base capacity/(CR1x γ_b) + (mean shaft capacity/ (CR1x γ_s)).

iii) In case if the minimum capacity is governing, then the design capacity of pile = (Minimum base capacity/ (CR2x γ_b) + corresponding shaft capacity/(CR2x γ_s))

or (Minimum shaft capacity/(CR2x γ_s) + corresponding base capacity/(CR2x γ_b)), whichever is less. The minimum of base and shaft capacities shall be taken for the same location. Calculated characteristic pile capacity shall be verified by conducting load tests.

4.5.3 *Calculation of Pile capacity from static load tests:*

Pile capacity can also be arrived by conducting static load tests. Number of load tests can be conducted. The capacity of pile shall be arrived based on settlement equivalent to 10% of pile diameter. This is called measured pile capacity. The mean of measured capacity shall be obtained. The minimum measured capacity shall also be identified.

The capacity of pile thus arrived shall be divided by the correlation factor to arrive at the allowable characteristic design resistance or capacity of pile. Depending upon the number of test locations, the correlation factor has been suggested. The correction factor shall be applied both on the mean capacity and on the minimum capacity. The correlation factor to be used is given in **Table 7**.

Table 7 Correlation Factors for Obtaining the Characteristic Capacity of Pile Foundation from Static Pile Load Test.

No. of tests	1	2	3	≥4
CR3	1.40	1.33	1.29	1.25
CR4	1.40	1.22	1.11	1.04

The characteristic capacity of pile Q_{uc} = Minimum of [Mean measured capacity/CR3, or Minimum measured capacity/CR4]

Design resistance of pile or pile capacity shall be obtained by dividing Q_{uc} by partial factor for resistance shown in **Table 5**.

$$Q_{ud} = Q_{uc}/\gamma_t \qquad \textbf{Eq. 8}$$

4.5.4 *Calculation of Pile Capacity from Dynamic testing using wave equation analysis.*

Dynamic tests shall be used to estimate the resistance of the ground provided adequate site investigations have been carried out in the form of boring and field tests. The test results have to be calibrated against static load test for characteristic capacity. Static load tests shall be conducted and all requirements shall be satisfied as given in Clause 4.5.3

The design value of the pile capacity is derived from the results of wave equation analysis on a number of representative piles. It shall be assessed as follows.

Characteristic capacity of pile Q_{uc}= Minimum of [Mean capacity/CR5, Minimum capacity/CR6]

Design resistance of pile or capacity of pile = Characteristic pile capacity/γ_t.

The value of CR5 and CR6 are shown in **Table 8**.

Table 8 Correlation Factors for Obtaining the Characteristic Capacity of Pile by Dynamic Testing

No. of tests	≥2	≥5	≥10	Remarks
CR5	1.94	1. 85	1.83	These factor shall be multiplied by a factor 0.85 if signal matching is carried out
CR6	1.50	1.35	1.30	

4.5.5 *Calculation of pile capacity from pile driving formula.*

The requirements as stated under 4.5.4 will be applicable for this method also. The CR factors stated in **Table 8** are also applicable with the following modifications. The values shall be multiplied by model factor 1.1 when the quasi – elastic pile head displacement is measured and 1.2 when it is not. **Appendix-7** of IRC: 78 (Part 1), method 2 can be followed for arriving at the resistances.

4.5.6 *Negative skin friction*

Piles which are taken through a fill to a suitable bearing stratum in the underlying natural soil or rock no support for compressive loads from shaft friction shall be assumed over the length of the pile shaft passing through fill due to downward movement of the fill as it compresses under its own weight or under the weight of the further surcharge placed over the fill area. This downward movement results in drag forces known as negative skin friction on the pile shaft. Clause 4.2 shall be referred to for the partial factor for this action. The partial factor on undrained shear strength given in **Table No. 2** for set 2 to be used for assessment of negative skin friction force for combination2 and seismic combination. For other combinations set 1 value shall be used. Method of application of partial factor on materials given in Clause 4.3.may be referred to

Using pile soil interaction, the depth of neutral point within this compressive layer shall be arrived which gives the length of pile subjected to negative skin friction. Alternatively as a simpler approach the entire length of the pile passing through the compressible layer can be taken as the length of pile subjected to skin friction. The negative skin friction force can be estimated to act over this length along the surface of the pile using the partial safety factor on undrained shear strength. For piles driving capacity predominantly from friction, the arrived downward drag force shall be multiplied by 0.25 and for piles which are predominantly end bearing, it shall be multiplied by 0.5 to arrive at the design force due to negative skin friction.

4.6 Group Action of Piles

4.6.1 *Piles subjected to compressive loads*

For the piles founded in clay soil, the group capacity may be estimated as given in Clause 709.3.3 (ii) of IRC:78. For block failure the base resistance and side resistance shall be estimated for each combination. The ground resistance factor given under open foundation shall be used to arrive at the base resistance capacity of the block which shall be added to the side resistance capacity to arrive at the total capacity of block. The material safety factor used for estimation of block capacity shall be same as used to estimate the capacity of individual piles. However, ground resistance factor 1.3 or 1.1 will be applicable given under open foundation. The partial factor for loads will be same as used for the calculation of loads on individual piles.

4.6.2 *Piles subjected to negative skin friction*

Piles passing through a fill will be subjected to negative skin friction effects. No support to compressive loads from the skin friction shall be assumed over this length. The skin friction on group of shall be estimated as follows. The lesser of the value (a) or (b) shall be used as negative skin friction effect on group of piles.

a) Number of piles x Negative skin friction effect on single pile
b) Plan area of the block enclosing all piles x length of pile as estimated in Clause 4.5.6 x density of soil in this block

4.7 Calculation of Pile Capacity for piles resting on Rock and Intermediate Geo-Materials

4.7.1 *Behavior of pile foundation resting on Rock:*

The load–penetration curve for rock of medium strength or less (≤ 100 Mpa) has a large plastic component despite the brittle nature of rock. The displacements required to mobilize the full bearing capacity of such rocks are very large and that a factor safety of 3 or 4 is required to limit the displacement to less than 2% of the diameter. Very brittle rocks (≥ 150 Mpa) do not exhibit plastic load -penetration curve and once the maximum strength is exceeded at any point in the brittle material total collapse occurs. Hence to limit the displacement, as such large displacements cannot be achieved at the site, the serviceability limit state approach has been suggested as a principal approach for design and verification by load tests. However only the capacity of pile at the ultimate state is required to be verified.

The allowable load on the pile under serviceability limit state shall be calculated from any of the following methods

(1) By calculation using the ground parameters obtained from site investigation

(2) Directly from static load tests

For method (1) static load tests need to be performed for correlation of pile capacity obtained by calculations

4.7.2 *Socketing length for pile*

The socket length for piles shall be provided as follows

(a) For hinged piles resting on rock proper seating has to be ensured. The minimum socket length should be 300 mm in hard rock and 0.5 times the diameter of the pile in weathered rock

(b) For moment carrying pile or pile fixed at base the length of socket shall be arrived as per Clause 9.2 of **Appendix -5** of Part1 of IRC:78

4.7.3 *Calculation of pile capacity using ground parameters from site investigation for serviceability limit state*

The ultimate axial load carrying capacity of the pile is determined by calculating the socket resistance and end bearing resistance separately based on the properties of rock obtained by laboratory testing or based in N value, as out lined in Clause 9.1 of **Appendix-5** of IRC:78. The ultimate capacity thus calculated shall be further divided by the relevant ground resistance factors 3.0 for the base resistance and 6.0 for socket resistance given in **Appendix- 5** of IRC:78 to obtain the allowable load on pile under Rare combination given in **Table B.3** of IRC:6. The end bearing component contribution after dividing by ground resistance factor shall be further limited to 5 Mpa. For calculation of socket friction capacity, the top 300 mm depth of rock shall be neglected. The frictional capacity shall be further limited to a depth of six times the diameter of pile. The displacement of pile under rare combination of load shall not exceed the limit specified in Clause 4.7.5.

4.7.4 *Allowable load on pile from static load tests*

The pile load test shall be conducted to satisfy the displacement limit specified in Clause 4.7.5. The load thus obtained from load tests shall be divided by correlation factor given in **Table 7**

of Clause 4.5.3 depending up on the number of tests conducted to arrive at the allowable load under rare combination.

4.7.5 *Allowable displacement under allowable load for rare combination*

The displacement under allowable load shall be limited to

For piles having diameter less than 600 mm -2 % of pile diameter but limited to 10 mm

For piles having diameter 600 mm or more -2 % of the pile diameter but limited to 18 mm

The displacement under the pile can be estimated theoretically by using IS:14593

4.7.6 *Verification of pile capacity under ultimate loads*

The characteristic capacity of pile using the partial factor on material set 1 value given in **Table 2** shall be computed. For calculating the characteristic capacity Q_u, Clause 9 .1 of **Appendix-5** of IRC:78 shall be used. The ultimate frictional and base resistances shall be calculated separately. Calculated resistances shall be further divided by the partial factors for ground resistance shown in **Table 9** and added to get the design capacity. The design capacity shall be compared with the axial forces applied on the pile.

GR3 resistance factors are to be adopted to estimate the reduced resistance of the pile under action combination 1. GR4 values are to be used for combination 2. GRS resistance factor shall be used for arriving at the reduced resistance under seismic combination. GRA shall be used for arriving at the reduced resistance under the respective accidental situation. In addition model factor of 1.15 shall be used along with the resistance factors to get the calculated design resistance of piles.

The allowable end bearing component after dividing by the resistance factor and modal factor shall be limited to 8.0 Mpa under all combinations. For calculation of socket friction capacity, the top 300 mm depth of rock shall be neglected. The friction capacity shall be further limited to depth of six times the diameter of pile.

Table 9 Partial Factor for ground Resistance for rock and intermediate geo materials

Component	Symbol	For bored and continuous augur flight piles		GRS for Seismic	GRA for Accidental Combination	
		GR3	GR4		For Vehicle collision, Log Impact and barge collision	Ship Collision
For Base resistance	γ_b	1.5	2.4	1.9	1.5	1.9
For Socket resistance	γ_s	3.0	4.8	3.9	3.0	3.9
For Combined resistance	γ_t	1.5	2.4	1.9	1.5	1.9

4.7.7 *Load test on piles*

Load tests on piles shall be carried out for arriving at the capacity or verification of capacity already calculated or for correlation/verification of capacity when the capacity is estimated by

methods other than load tests. The test pile will be of same type, length, cross section and comparable soil parameters. Reinforcement in pile shall be as provided in actual pile. Piles will be subjected to characteristic loads for piles resting on soil and to allowable load under rare combination in case of piles resting on rock.

4.7.8 *Group action of Piles*

There will not be any block failure for piles founded on rock. The capacity of group shall be taken as total capacity of the piles in the group.

4.8 Verification of Tensile Resistance of Pile Foundation

4.8.1 *Combination of loads:*

In order to arrive at the design tensile resistance of an isolated pile, the load combinations as per **Table B.4** of IRC:6 shall be followed. However the partial safety factor for actions shall be modified as follows.

Permanent actions resisting the tensile force, the partial factor shall be taken as 1.0. These actions are considered favorable actions in all combinations. However, if the permanent actions increase the tensile force, the same may be considered with the partial factors as per the **Table B.4**. For buoyancy the partial factor shall be taken as 1.0.

4.8.2 *Resistance capacity of single pile:*

The tensile resistance capacity of the pile is dependent on the friction around the shaft and the weight of pile. The Buoyant weight of pile is calculated with partial factor as 1.0. The shaft tensile resistance of single pile shall be calculated using the partial factor on material properties of set 1 value shown in **Table 2** for all the combinations. The frictional resistance offered by shaft can be assessed by any of the following methods

(a) By direct calculation from the ground parameters obtained from site investigation

(b) By carrying out pull out tests

(c) By calculating from ground profile test results

4.8.3 *Calculation of shaft tensile resistance of pile in soil:*

4.8.3.1 *Calculation of shaft tensile resistance from ground parameters*

The shaft frictional resistance Q_{ufc} offered by the pile shall be calculated using Clause 2 of **Appendix-5** of IRC:78.

This shall be treated as characteristic shaft tensile resistance of pile Q_{tk}. This characteristic resistance shall be further divided by partial factor for ground resistance and model factor given in **Table 10** for the design shaft tensile resistance of pile Q_{td}, which may be calculated as below:

Design shaft tensile resistance, $Q_{td} = Q_{tk}/(\gamma_{st}\, M)$ **Eq. 9**

Table 10 Partial Factor for Resistance for Tension or uplift Piles in soil

Resistance	Symbol	GR5 For combination 1	GR6 For combination 2	GRS for Seismic	GRA for Accidental Combination		Model Factor M
					For Vehicle collision, Log Impact and barge collision	Ship Collision	
Shaft in Tension	γ_{st}	1.0	2.0	1.60	1.25	1.60	1.5

4.8.3.2 Calculation of shaft tensile resistance from pull out test:

Pull out tests will be carried out at number of locations. The mean value of resistance shall be calculated.

The characteristic shaft tensile resistance Q_{tk} shall be Minimum of [Mean resistance/CR3, and Minimum Resistance/CR4]

CR3 and CR4 are the modified partial factors as 1.1 times values mentioned in **Table 7**.

The design shaft tensile resistance Q_{td} = characteristic tensile resistance/partial factor for ground resistance γ_{st}

Partial factor for ground resistance shall be taken from **Table 10**. The model factor shall not be used

4.8.3.3 Calculation of shaft tensile resistance from ground profile test result (eg. Cone penetration)

The shaft resistance shall be calculated using the ground parameters from the field tests. Reference can be made to Clause 7 of **Appendix-5** of IRC:78 for the calculation of shaft tensile resistance. Resistance thus obtained is termed as calculated resistance. The characteristic resistance shall be as given below.

Q_{tk} characteristic shaft tensile resistance = Minimum of {Calculated mean resistance/CR1; Minimum tensile resistance/CR2}

CR1 and CR2 values are modified partial factors, as 1.1 times the values as per **Table 6**.

The Characteristic shaft tensile resistance shall be divided by partial factor for ground resistance γ_{st} as per **Table 10** to arrive at the design shaft tensile resistance. Q_{td}. Model factor shall not be used. The design shaft tensile resistance may be worked out by formula given below:

$$Q_{td} = Q_{tk}/\gamma_{st} \qquad \textbf{Eq. 10}$$

4.8.3.4 Total Uplift Resistance of piles or group of piles in soil:

The Total uplift Resistance of a Single Pile (TRSP) will be equal to tensile resistance of single pile + weight of pile as appropriate. The uplift resistance of single pile shall be calculated as mentioned above.

The uplift resistance of group of piles when all of them are subjected to tensile force shall be calculated as per Clause 709.3.6.3. of IRC:78 Part-1.

The shear resistance mobilized on the surface of perimeter shall be calculated using the partial factors shown under set 1 on materials as per **Table 2** for all combinations. The partial factor on weight of soil contained in the block and self weight of pile shall be taken as 1.0. The tensile resistance of group of piles subjected to tension or uplift shall be the lower of the sum of tensile resistance of the capacity of individual piles in the group or the block resistance calculated as above

4.8.3.5 Ultimate design uplift capacity of piles in rock or intermediate Geo-materials

The top 300 mm depth of the socket may be omitted for calculating the uplift capacity using ground parameters. The ultimate socket resistance shall be calculated as per Clause 9 of **Appendix-5** of IRC: 78 . The ultimate socket resistance shall be divided by the ground resistance factor for socket resistance which may be taken from **Table 9** and model factor 1.15 to obtain the design resistance. The length of the socket shall be limited to 6 times the pile diameter. The frictional resistance capacity of soil above the socket shall be calculated as per Clause 4.8.3.1 and added to socket resistance capacity to arrive at the design ultimate pull out capacity of pile with socket in rock. The self-weight of pile shall also be added. The design pull out capacity may also be estimated from pull out tests. The correlation factor for pull out test shall be as per **Table 7** and multiplied by factor 1.1 shall be used to estimate the characteristic pull out capacity from tests. The method as per Clause 4.8.3.2 shall be followed to work out the design uplift capacity using the correlation factor and socket resistance factor as per **Table 9** from the field test results. The total uplift resistance of block shall be tensile capacity of all the piles in the group. Which is as follows:

(Pile capacity of single pile × Number of piles).

4.9 Settlement and Serviceability Limit State:

Piles resting in medium to dense soils, the partial factor for ultimate design have been so chosen that occurrence of serviceability limit state in the supported structure is prevented. Thus, the settlement check is not required. However if settlement is required of the soil is not medium to dense or the soil is a cohesive, the settlement may be calculated as per Clause 709.3.4 of IRC:78 for quasi permanent load combination given in **Table B3** of IRC:6 using IS 8009 Part 2.

4.10 Routine Load Test

The piles resting on soil shall be tested for 1.5 times of the rare combination of loads calculated using **Table B.3** of IRC:6. The settlement shall not exceed the settlement arrived based on shear parameters and settlement parameters. The pile resting on rock shall be tested for rare combination of load and the settlement shall satisfy Clause 4.7.5

4.11 Piles Subjected to Lateral Load

4.11.1 *Ultimate horizontal capacity of pile*

The design lateral Capacity of a pile shall be calculated and checked against all combinations of factored loads shown in **Table B-4** of IRC:6. For arriving at the characteristic lateral capacity the

partial factor on material, Set 2 values as per **Table 2** shall be used to estimate the geo technical forces and lateral resistances for combination 2, for seismic combination and for accidental combination. For combination 1 partial factor for set 1 value shall be used. No ground resistance factor needs to be applied in lateral direction. The characteristic lateral capacity thus arrived shall be multiplied by partial factor shown in **Table B-4** of IRC:6 to arrive at the design capacity

The ultimate horizontal capacity of the short pile shall be estimated by using Brinch Hansen's method or Broms method for which specialist literature (pile design and construction by M. J. Tomlinson or H.G. Poulos or any other relevant literature) may be referred to. The piles will be treated as fixed head when pile caps are provided.

In case of very long piles the passive resistance provided by the lower part of soil pile is quite large. In such case the ultimate horizontal load which can be carried by the pile is determined solely from the moment of resistance of the pile. Alternatively piles can be analyzed using special software applicable to pile foundations taking care of the ultimate behavior of soils. It shall be ensured that the resistance capacity of pile worked out shall be greater than the requirement

4.11.2 *Ultimate horizontal capacity of piles in a group*

The Design horizontal ultimate capacity of group of piles shall be arrived by calculating the passive resistance using the Set 2 material safety factor given in **Table 2** for combination2, seismic combination and accidental combinations, and Set 1 values shall be used for combination1 over a wall depth equal to 6D and width equal to L+2 B where L,D and B are defined in Clause 709.3.5.1 of IRC:78 and multipled by partial factor on actions as given In **Table B.4** of IRC:6.

However the lateral resistance of a pile group must not exceed the sum of lateral resistance of individual piles

4.11.3 *Horizontal capacity of pile under serviceability limit state:*

The single pile resistance capacity shall be arrived by conducting lateral load test. The lateral deflection shall not exceed 1% of pile diameter at scour level. In case of land piles where there is no scour it shall be measured at pile cut off level *i.e.*, at bottom of pile cap. The capacity shall not be less than the capacity requirement for Rare combination of actions as per **Table B-3** of IRC:6. For other combinations no test need to be conducted.

4.12 Structural Analysis of pile:

The ultimate moment, obtained from the horizontal capacity worked out by any one of the above methods shall be proportioned to the ultimate horizontal load applied on the individual pile to obtain the design ultimate moment in the pile.

For serviceability limit state check, the moment in the pile can be obtained by treating the pile as an elastic beam supported on soil springs using modulus of sub grade reaction or by using Matlock and Reese method or by Broms method or equivalent cantilever method which is applicable to light loading. The deflection shall be computed by using the above methods or the method giving by Poulos and Davis.The equivalent cantilever method is an approximate solution, applicable for calculation of deflection due to small loads.

4.13 Structural Design of Pile

Structural design of pile shall be carried out as per IRC:112 for both ultimate limit state and serviceability limit state.

For U.L.S Verification, combinations of actions as per **Table B-4** of IRC:6 along with the relevant partial factors for geo-technical materials shall be used to estimate the geo-technical forces.

For S.L.S verification, combination of actions as per **Table B-3** of IRC: 6 with partial factor on geo-technical materials as 1.0 shall be used to estimate geotechnical forces.

4.14 Special Vehicle Loading:

4.14.1 *Verifications Required*

Verification of pile capacity and strength checks are required to be carried out under ULS. The partial factor of 1.15 as per Clause 204.5.4 IRC: 6 shall be applied for SV loading for combination 1 and the same shall be 1.0 for combination 2.

4.14.2 *Method of Verification*

All the relevant clauses for other loadings shall also be applicable for verifications with the following modifications:

(a) The ground resistance factors (GR) shall be reduced by 15% when using combination 2 for piles resting on soil when subjected to compressive loads.

(b) The ground resistance factor shall be taken as 1.2 for calculation the group block capacity of piles as per Clause 4.6.1 for piles resting on soil and subjected to compressive loads.

(c) The factored ultimate resistance of piles computed as per Clause 4.7.6 for combinations 1 and 2 shall be further increased by 1.25 times to arrive at the ultimate capacity under this loading for piles resting on rock.

4.15 Other Provisions

Any provisions not covered in these sections shall be as per section 709 of IRC:78

5. WELL FOUNDATIONS

5.1 Limit State Verification of Side Earth Resistance and Bearing Resistance

The side earth resistance and bearing resistance of well foundation shall be verified for all the four combinations outlined in **Table B.4** of IRC:6.

The equilibrium check shall be carried out by using partial factor for material given in **Table 1** and for loads given in **Table B.1** of IRC: 6 and resistance factors given in Clause 5.3 of this Code. At first the partial factor on materials (given in **Table 1** for equilibrium check and in **Table 2** for foundation design of this Code) shall be applied and the active and passive pressure forces shall be calculated. Next partial factor as per **Table B.1** for equilibrium check and **Table B.4** of IRC:6 for foundation design shall be applied on these earth pressure forces (on active and passive

earth pressure forces). Finally, the calculated earth side resistance force and moment, both shall be further divided by side resistance factor given in Clause 5.4 to obtain the design earth side resistance and moment.

Similarly the characteristic base resistance shall be calculated by using the respective partial factor on materials at first. The design base resistance shall be obtained by further dividing the characteristic resistance by respective base resistance partial factor given in Clause 5.5.

5.2 Loads and Load Combinations

The following four combinations as per **Table B.4** of IRC: 6 shall be considered.

(a) Load Combination 1
(b) Load Combination 2
(c) Seismic Combination
(d) Accidental Combination

The partial factors for loads shall be as per **Table B.4** of IRC: 6. All the loads therein shall be combined appropriately. For tilt and shift effects, the respective partial factor for loads shall be used to arrive at the tilt and shift moment.

The principle of combinations is as follows:-

Load combination 1 + Partial factor on Material Set value 1+ Resistance factor for side resistance and base resistance

Load combination 2 + Partial factor on Material Set value 2 + Resistance factor for side resistance and base resistance

Seismic combination + Partial factor on Material Set value 2 + Resistance factor for side resistance and base resistance

Accidental combination + Partial factor on Material Set value 2 + Resistance factor for side resistance and base resistance

Meaning of "+" combined with

5.3 Partial factor for geo-technical Material for Computation of Resistances

Two sets of partial factors for materials (Set 1 and Set 2) are shown in **Table 2**. The active and passive pressure co-efficient shall be calculated using Clause 1 of **Appendix-3** of IRC:78 using partial factor on material

The partial factors for Set 1 value shall be used to evaluate the geo technical force to check Load Combination 1 and partial factors for Set 2 shall be used for remaining combinations. The partial factors on materials shall be used both for arriving at the side earth resistance as well as the base resistance. The side earth resistance shall be ignored in case of foundations when resting on rock having a unconfined compressive strength more than 6 Mpa.

5.4 Resistance Factors for Side and Base Resistance and Limitation of Side Resistance

5.4.1 *Resistance factors for side and base resistance:*

The partial factor for side resistance shall be 1.0 for all combinations

The partial factor for ground resistance (bearing capacity) shall be as in **Table 11**.

5.4.2 *Limitation of side resistance*

The following requirement shall also be satisfied for the combinations shown in **Table B.4** of IRC:6 for the design of foundation. The ratio of Characteristic net side resistance (P_p-P_a)/design net side resistance (P_p-P_a) shall not be less than 1.9 for combination 2 and 1.5 for seismic and accidental combinations. The Characteristic net side resistance shall be calculated by using both partial safety factor for geo technical force and partial factor on materials as 1.0. For calculating the design net side resistance the relevant partial factor on geo technical force and materials shall be considered as explained in Clause 5.1.

5.5 Summary of Resistance Factor for Base Resistance

The base resistance factors as per **Table 11,** applicable for foundations resting on soil

Table 11 Summary of Base Resistance Factor for all Combinations

Combination	Action Combination (1)	Action Combination (2)	Seismic Combination	Accidental Combination
Partial Factor for Materials for Base Resistance	Set 1	Set 2	Set 2	Set 2
	1.35	1.10	1.0	1.0

5.6 Verification of Base Resistance and Base Contact Area

5.6.1 *Wells resting on soil*

The base pressure will be verified for all the combinations shown in **Table B.4** of IRC: 6 with the appropriate partial factor on material and resistance. A minimum 80% of contact area of base shall be ensured under all combinations. Linear triangular pressure distribution shall be assumed for calculation of base pressure. The allowable bearing pressure shall be further restricted to 3.0 MPa.

5.6.2 *Wells resting on rock*

The base pressure shall be verified for all the combinations shown in **Table B.4** of IRC: 6. The appropriate partial factor on materials shall be used. The base resistance capacity shall be based on the crushing strength of rock with resistance factor of 4.5 for all combinations.

The contact area of base shall not be less than a minimum of 80% of the base for combinations 1 and 2 and 67% for seismic and accidental combinations. Linear pressure distribution shall be assumed.

5.7 Verification of Serviceability Limit State

The differential settlement between the adjacent foundations shall be verified by using Quasi permanent combination. The angular distortion of superstructure resulting on account of differential settlement shall not exceed 1. in 400. The total settlement shall also be calculated as specified under Clause 3.4.2 using IS 8009 part 2.

For calculation of base pressure the characteristic net side resistance shall be divided by 2.0 to arrive at the design net side resistance. This reduced net side resistance shall be used in the calculation. The partial factor on material shall be taken as 1.0 for this purpose.

The base contact area shall be 100% for foundations resting on soil and at least 80% for foundations resting on rock under frequent combination of live load. The base pressure shall be estimated under rare combination of loads. For foundations resting on rock, a factor of safety of 7 on unconfined crushing strength of base rock shall be used to arrive at allowable base pressure. For foundations resting on soil, a factor of safety of 2.0 on ultimate bearing capacity shall be used. The bearing resistance. shall be restricted to 1.5 MPa.

5.8 Structural Design of Foundation

The structural design of foundation shall be carried out as per IRC:112 for all the combinations as per **Tables B.3** and **B.4** of IRC:6. Due considerations shall be given to material safety factor for geo-technical materials while computing the effects of geo technical forces.

5.9 Special Vehicle Loading

5.9.1 *Verifications required*

Verification of Equilibrium, base pressure and strength checks are required to be carried out under ULS. Verification of base pressure and strength checks for rare combination of loads under SLS shall also be carried out. The partial factor on SV loading, for combination 1 and Equilibrium check shall be taken as 1.15 as per Clause 204.5.4 of IRC:6 for ULS verifications. For verifications under combination 2 under ULS and rare combination under SLS, the partial factor on SV loading shall be taken as 1.0.

5.9.2 *Method of Verification*

All the relevant clauses for foundation shall be applicable for carrying out the verifications with the following modifications:

1) The base resistance factor shall be taken as 1.20 for combination 1 and 1.0 for combination 2 as against 1.35 and 1.10 mentioned in Clause 5.5 for arriving at the allowable bearing resistance for foundations resting on soil.
2) Factor of safety of 6.0 shall be ensured as against 7.0 mentioned in Clause 5.7 for arriving at the allowable bearing pressure while checking the base pressure under rare combination of loads for foundation resting on rock. For foundation resting on soil, the factor of safety of 1.6 shall be maintained with respect to ultimate bearing resistance under rare combination of loads.

5.10 Other Provisions

Well foundations designed by the Limit State Approach given in this code need not be checked using IRC: 45. Any provision not covered in this code shall be governed by provisions of Clause 708 of IRC: 78.

EXPLANTORY NOTE
1. LIMIT STATE DESIGN APPROACH AND DESIGN OF OPEN FOUNDATION

1.1 Introduction

This note gives the limit state approach adopted in Europe and explained how proposed draft clauses have been framed. AASHTO LRFD Provisions and articles written by several authors have also been taken into account consideration while drafting the code. 'Pile foundation design and construction practice' by M.J Tomlinson and 'Pile foundations analysis and design' published by H.G Poulos have also been followed. Some of the provisions of exisiting code have been followed but converted to limit state approach. Design of foundations for SV loading also has been included

It is explained, when combination 1+ partial factor on material set value1+ ground resistance factor R1 has been mentioned, it should be interpreted as follows:

Use partial factor and combination of structural actions as mentioned for combination 1 in **Table B.4** of IRC:6.

Calculate the geotechnical actions, reactions and ground resistances applying the partial safety factor on geotechnical materials (to calculate the actions, reactions and resistances) shown under set value1 given in **Table 2** of the code.

The calculated geotechnical actions and reactions shall be further multiplied by respective partial factors shown in **Table B.4** of IRC:6 and combined with factored structural actions. The calculated ground resistances shall be further divided by Ground resistance factor R1 to arrive at the allowable ground resistance under this combination of structural and geotechnical actions. Other combinations may be interpreted similarly.

1.2 Adoption of Partial factor on geo technical Material for Equilibrium Check

The current day practice is to adopt material safety factors both for soil and rock so that the values obtained from lab or field tests (treating it characteristic strength,) the design strength is obtained. This is similar to partial safety factor on concrete and reinforcement. The partial safety factors for checking the equilibrium is different from the partial safety factors for designing the foundation. These are given in **Table 1** and **Table 2** of the code.

1.3 Check for overturning and sliding

1.3.1 *Check for overturning Partial Factor on Earth Pressure action = 1. 5 as per **Table B.1** of IRC:6*

Applying the material safety factor as given in **Table 1** of the code Increase in Earth Pressure 1.12 times due to reduction in ϕ

Partial Factor on Stabilizing force = 0.9 as given in **Table B.1** of IRC:6

FOS against overturning after taking into account the above factors is 1.5 x 1.12/0.9= 1.87 which is approximately 2.0. Same as given in the present code

Resisting moment = 1.87 times overturning moment. In seismic condition the factor is 1.5/0.9 =1.67 times overturning moment greater than 1.5 as per our present code

1.3.2 *Check for sliding*

$H_d \leq R_d$

H_d is the sliding force and R_d is the resisting force.

Partial factor on Earth pressure 1.50.

Increase in Earth Pressure as given above 1.12

Reduction in Tan δ value due to reduction in ϕ= 0.92

In the present code1.5 is the factor of safety against sliding. Resisting force required when material safety factor 1.0 is adopted $\frac{1.5}{0.9\tan\delta} = \frac{1.66}{\tan\delta}$

Tan δ taken as 1.0 for comparison.

Factor against sliding =

If no geo technical action is involved then 1.5/0.9X0.92 =1.8 (seismic case).

In earth pressure case 1.5x1.12/0.9x0.92=2.02

ie 1.66 H ≤ V tan δ or 2.02 H ≤ V tan δ Increase level from present 1.5 1.10 1.30

1.4 Explanation to the pressure diagram

The pressure diagram shown is as per clause and commentary of 10.6.1.3, C.10.6.1.3, 10.6.1.4 and 11.6.3.2 of AASHTO LRFD 2012 edition and also as per the article "How to design structures using Euro code 2 foundations" by R. Webster and O. Brooker Published by Concrete Center of the UK. For foundations resting on soil, the principle is the centre of gravity of reduced effective area is always concentrically loaded (i.e). The C.G of effective area and the location of applied load should coincide (AASHTO) so that the foundation will have uniform pressure under ULS condition.

This method is also given IS:6403. For foundations resting on rock triangular or trapezoidal shape of diagram has been recommended. For SLS condition the pressure diagram will be either triangular or trapezoidal for both these foundations as being followed at present. For structural design of foundation the shape of diagram will always be linear. All recommendations are from AASHTO LRFD and some recommended by several authors.

1.5 Discussion on Base contact area

A) Rectangular Foundation:

Table 1 Eccentricity V's the base area in contact

Sl. No	e/B	For Uniform Pressure distribution			For Triangular Pressure distribution		
		Contact area in percentage	Max Pressure coefficient	Base width of contact in percentage	Contact area in percentage	Max Pressure coefficient	Base width in contact In percentage
1	0.10	80	1.25	80	100	1.60	100
2	0.15	70	1.43	70	100	1.90	100
3	0.166	67	1.49	67	100	2.0	100
4	0.20	60	1.66	60	90	2.22	90
5	0.23	54	1.85	54	81	2.47	81
6	0.25	50	2.0	50	75	2.66	75
7	0.33	34	2.94	34	51	3.92	51

B) Circular Foundation

Table 2 Eccentricity V's the Base Area in Contact

Sl. No	e/d	For uniform pressure distribution			For Triangular Pressure distribution		
		Contact area in percentage	Max Pressure coefficient	Base width in contact percentage	Contact area in percentage	Max Pressure coefficient	Base width in contact percentage
1	0.05	87	1.15	82			
2	0.75	79	1.20	75			
3	0.10	76	1.31	71			
4	0.125	70	1.43	65	100	2.0	100
5	0.15	64	1.56	60	93	2.23	91
6	0.20	50	2.0	50	80	2.76	76
7	0.25	39	2.56	41	65	3.55	61.5
8	0.275	32	3.13	37	57	4.15	55.0
9	0.300	28	3.57	32	50	4.96	48.5
10	0.40				19	13.87	23.5

1.6 Fixation of Minimum allowable base contact area

To fix the minimum allowable area of base in contact, combination 2 under ULS which is a critical condition and frequent combination under serviceability limit state have been taken up with different Dead Load and Live load ratios to arrive at a conclusion. Dead load live load ratios considered are 50:50 60:40 and 70:30. Combination 2 is chosen as it will be critical combination for arriving at the size of foundation as it is similar to service load combination

A) Foundation resting on Soil Rectangular Footing:

Table 3 Base area in contact and base pressure

Description	DL:LL 50:50		DL:LL 60:40		DL:LL 70:30	
	Combination 2	Serviceability limit state frequent load combination	Combination 2	Serviceability limit state frequent load combination	Combination 2	Serviceability limit state frequent load combination
Dead Load (Total Load P)	0.5P	0.5P	0.6P	0.6P	0.7P	0.7P
Live Load 0.5 x 1.3P	0.65P	0..375P	0.52P	0.3P	0.39P	0.225P
0.4 x 1.3=0.52						
0.3x1.3 =0.39						
0.5 x 0.75=0.375						
0.4 x 0.75=0.3						
0.3 x 0.75-0.225						
Total Load	1.15P	0.875P	1.12P	0.9P	1.09P	0.925P
Moment 1.3x1.25 x M	1.625M	1.00M	1.625M	1.0M	1.625M	1.0M
(PF x variation in property						
e =	1.41 (M/P)	1.14 (M/P)	1.45 (M/P)	1.11 (M/P)	1.49 (M/P)	1.08 (M/P)
Assume 50% base contact at ULS	50	-	50	-	50	-
For this contact e = for rectangular foundation	0.25B		0.25B		0.25B	
Corresponding e for serviceability limit state (0.25 x 1.14/1.41)	-	0.20B	-	0.20B	-	0.18B
Corresponding contact area at SLS	-	90%	-	90%	-	96%
Maximum Pressure	2x1.15 = 2.30	2.22x0.875 = 1.94	2x1.12=2.24	2.22x0.9 = 2.0	2x1.09 = 2.18	2.08x92 = 1.924
(Maximum pressure will be resisted with a factor of safety 2.0 on ULS capacity	2.0		2.0		2.0	
FOS at SLS		$\frac{2x2.3}{1.94} = 2.37$		$\frac{2x2.24}{2.0} = 2.24$		$\frac{2x2.19}{1.924}$ =2.28
Conclusion 50% contact at ULS will lead to 90% of contact at serviceability limit state. FOS for bearing pressure ULS=2.0 FOS for bearing pressure at serviceability limit state not be less than 2.25						

(B) Foundations resting on soil – Circular footing

Table 4 Base area in contact and base pressure

Description	DL:LL 50:50		DL:LL 60:40		DL:LL 70:30	
	Combination 2	Serviceability limit state frequent load combination	Combination 2	Serviceability limit state frequent load combination	Combination 2	Serviceability limit state frequent load combination
Circular Foundation						
e for Circular foundation for 50% contact	0.20d		0.20d		0.20d	
Base contact at ULS	50%		50%		50%	
Corresponding for serviceability limit state	-	0.16d	-	0.16d	-	0.144d
Corresponding base contact at SLS	-	91		91		93
Maximum 10% Loss of Contact of Base						
Maximum base pressure	2x1.15=2.30	2.33x8.75 =2.04	2x1.12=2.24	233.09=2.09	2x1.09=2.18	2.23x925 =2.06
FOS at SLS		2x2.30/2.04 =2.25		2x2.24/2.09 =2.14		2x2.18/2.06 =2.12
FOS average will be 2.17 This proposal can be adopted (i.e) 50% contact at ULS to 90% contact at serviceability limit state FOS at ULS 2.00 at SLS = 2.17						

(C) Checking of Rectangular Foundation for Factor of Safety against Bearing Capacity for Combination 1 and Rare Combination

Table 5 Checking of Rectangular Foundation for Factor of Safety against Bearing Capacity for Combination 1 and Rare Combination

Description	DL:LL 50:50		DL:LL 60:40		DL:LL 70:30		Remarks
	Combination 1			Rare Combination			
DL: LL Ratios	Combination 1 50:50	Combination 1 60:40	Combination 1 70:30	Rare Combination 50:50	Rare Combination 60:40	Rare Combination 70:30	
Dead Load (Total Load P)	0.675P	0.81P	0.945P	0.5P	0.6P	0.7P	
Live Load 0.5 x 1.5 P	0.75P	0.60P	0.45P	0.50	0.4P	0.3P	
Total Load	1.425P	1.41P	1.39P	1.0P	1.0P	1.0P	
Moment 1.5M	1.5M	1.5M	1.5M	1.0M	1.0M	1.0M	
e = M/P	1.5/1.42 x M/P = 1.052	1.063	1.07	1.0	1.0	1.0	

For 50% at ULS							
e/B =e for comb A1/A2xratio width of contact of base	1.0.5/1.41 x .25	1.0.6/1.45 x .25	1.0.5/1.49 x .25				
new e//B	0.186	0.182	0.180	0.177	0.172	0.168	
Base % CONTACT	63	63.70	64.11	96.76	98.82	99.41	
Maximum pressure coefficient	1.59	1.57	1.56	2.07	2.04	2.01	
Maximum Pressure	1.425 x 1.59 = 2.26	2.21	2.17	2.07	2.04	2.01	
Factor of Safety	2 x 2.30 / 2.26 = 2.035	2 x 2.24 / 2.21 = 2.03	2 x 2.18 / 2.17 = 2.00	2 x 2.3 / 2.07 = 2.22	2 x 2.24 /2.04 = 2.20	2 x 2.18 /2.01 = 2.17	

Conclusion:	**Combination 2**			**Combination 1**			**Rare Combination**			**Frequent Combination**		
Ratio of DL:LL	50 50	60 40	70 30	50 50	60 40	70 30	50 50	60 40	70 30	50 50	60 40	70 30
Base Contact area	50	50	50	63	63.70	64.01	97	99	99.50	90	90	96
FOS against bearing capacity	2.0	2.0	2.0	2.04	2.03	2.0	2.22	2.20	2.17	2.37	2.20	2.28

Note: Combination 2 is critical for size of foundation: Both for contact are and base pressure. If 2 is satisfied all other combination will be automatically satisfied.

(D) Foundation Resting on Rock

Linear distribution to be adopted. Adopt 80% contact at SLS.

Table 6 Rectangular foundation. Triangular distribution – on Rock

Base area at SLS	-	80%	-	80%	-	80%
Ratio ULS/SLS	1.23	-	1.30	-	1.38	-
e/b ratio	0.28	0.23	0.29	0.23	0.32	0.23
Base in contact	66	80	63	80	54	80
Revise base Contact to 67% at ULS e=	0.277	0.225	0.277	0.220	0.277	0.200
Base in contact both in ULS and SLS	67%	83%	67%	84%	67%	90%
Max pressure FOS at ULS adopting FOS 7 in SLS	2.98x1.15 =3.43 7x2.1/3.43 =4.3	2.40x875 =2.10 7	2.98x1.12 =3.34 7x2.14/3.34 =4.48	2.38x0.9 =2.14 7	2.98x1.09 =3.24 7x2.04/3.24 =4.40	2.22x.925 =2.04 7

FOR CIRCULAR FOUNDATION e/d for Circular Foundation for 67% contact at ULS	0.24	0.24/1.23 = 0.195	0.24	0.24/1.26 =0.19	0.24	0.24/1.38 =0.174
% of base contact	67	80	67	83	67	87
Max pressure	3.39x1.15 =3.90	2.76x0.875 =2.41	3.39x1.12 =3.80	2.65x0.9 =2.38	3.39x1.09 =3.70	2.5x0.925 =2.317
FOS	4.3	7	4.38	7	4.35	7
For rock 67% contact at ULS will lead to More than 80% contact in service.						

Based on the above calculation it is recommended to have the following % of contact of base.

Table 7 Recommendation of Base Contact Areas

	Under ULS	Under SLS
In Soil	50%	90%
I Rock	67%	80%

If shall be noted if we satisfy one condition the other condition will be automatically satisfied As the limit state code is being put into use for the first time for foundations, after gaining confidence SIS check for contact area and bearing pressure can be dispensed with at a latter stage.

1.7 Allowable Bearing Pressure

For calculation of allowable bearing pressure(resistance) the partial factor on material is to be taken as 1.0 for combination 1, and accidental combination

Hence whatever the capacity is worked out shall be used. The average partial factor on action will be approximately 1.45 in case of combination 1.

In order to be very close with the existing code provisions on factor of safety on bearing resistance, a Resistance factor to the ground resistance can be further introduced. Using this provision by introducing resistance factor 1.3, the FOS will workout ≈2.0. (1.45x1.3) for combination 1 and 1.3 for accidental combination.

For combination 2 and the seismic combination, the partial factor on material has to be used. Hence the reduction in main contribution to bearing capacity is from Nq. For reduction in Nq Refer sensitivity Table under the well foundation. It will be 60% after adopting partial factor on material which will lead to factor of safety of 1.66. In order to have safety factor 2.0 the resistance factor can be 1.30.for combination 2.

Hence the resistance factor 1.3 is recommended for the 3 combinations. (1, 2 and Accidental). For seismic combination the factor will be 1.1 (1.66X1.1=1.8).

The size of footing will be governed by combination 2 and the structural design will govern by combination 1.0.

This will ensure a factor safety of 2.25 on bearing capacity at serviceability limit against the ULS capacity. In case of rock resistance factor of 4.5 shall be taken in all combinations

In case of Rock the factor of safety at ULS for combination 2 will be 6.3 (4.5x1.4) which will lead to factor of safety at SLS more than 7.0 (Which are more are less same as the existing code)

1.8 Serviceability Limit State Check

The contact area requirement shall be satisfied using frequent combination. The bearing pressure need to be checked under rare combination. The combination 2 represents almost serviceability limit state. The factor of safety automatically will be 2.25 and for rock 7.0... Settlement calculation if required has to be carried out using Quasi Permanent Combination and Partial safety factor on material shall be taken as 1.0.

1.9 Final Recommendations

(a) For checking the equilibrium the material safety factor as suggested shall be adopted.

(b)

Base in contact (Soil)	At ULS	50%
	At SLS	90%
Base in contact (Rock)	At ULS	67%
	At SLS	80%

(c) Ground Resistance factor for 3 combinations is 1.3 and for, Seismic Combination 1.1 FOS on ULS bearing capacity 2.0 and 2.25 in Serviceability Limit State for soil.

For rock a factor of safety of 4.5 at ULS will lead to FOS of 7 at SLS

(d) Serviceability Limit State

Bearing Pressure and base contact need be checked at Serviceability limit state. The settlement if required can be calculated using Quasi or Semi Permanent Combination.

1.10 Factors for SV loading

1.10.1 *Analysis of Safety factor on Soil for SV loading for 50% DL: LL ratios*

A) Normal Case of loading

Partial factor on actions in combination 2 =(1+1.3)/2 = 1.15

Material Safety Resistance factor is taken 1.538 for all ϕ values

(PFAx GRx PFM)

Combination 2 total FOS: DL: LL 50:50 = 1.15 x 1.3 x 1.538 = 2.30

DL: LL 70:30 = 1.09 x 1.3 x 1.538 = 2.17

Average: 2.20

On Ground Strength FOS= 2.0

Partial factor on actions in combination 1= (1.35+1.5)/2 = 1.425

Combination 1 total FOS DL:LL 50:50 = 1.425 x 1.30 x 1.0 = 1.85

DL:LL 70:30 = 1.395 X 1.30 x 1.0 = 1.81

Average: 1.83

On Ground Strength FOS= 1.30

B) **With S.V loading**

For Combination 2 total FOS : 1.0 x 1.2 x 1.538 = 1.85, (1.85/2..2x100 = 84%) on Ground strength = 1.85

For Combination 1 total FOS: DL: LL 50:50 = 1.25 x 1.2 x 1.0 = 1.5, (1.5/1.83x100 = 82%) on Ground strength = 1.20

DL: LL 70:30 = 1.29 x 1.20 = 1.55, (85%)

For DL: LL 50:50 PFA = (1.35+1.15)/2 = 1.25

For DL: LL 70:30 PFA = (1.35 x 0.70 + 1.15 x 0.30) = 1.29

So adopt 1.2 for both the combinations for SV loading for ground resistance

Analysis of safety factor for Rock Foundation for SV loading

Combination 2 for Normal loading: DL: LL 50:50 = 1.15 x 4.5 x 1.4 = 7.245

DL: LL 70:30 = 1.09 x 4.5 x 1.4 = 6.86 Average 7

For Combination 1 DL: LL 50:50 = 1.425 x 4.50 = 6.40

DL: LL 70:30 = 1.395 x 4.50 = 6.27 Average 6.35

Now with SV loading For Combination 2 = 1.0 x 4.5 x 1.4 (6.3) (90%)

For Combination 1 DL:LL 50:50 = 1.25 x 4.50 = 5.6

DL:LL 70:30 = 1.29 x 4.50 = 5.8

Hence retain resistance factor of 4.5 given for other loading for SV loading also

2. DESIGN OF PILE FOUNDATION

2.1 Material Factor adopted for checking Equilibrium:

Values as given in **Table 1**

2.2 Partial factor on material for checking geotechnical capacity and Geo technical actions

Partial factor on material set 1 and set 2 values as given in **Table 2**.

2.3 Ground Resistance Factor:

Use of partial factors for ground resistances have been suggested so that the load carrying capacity of the pile is at least close to the capacity predicted by the existing code of practice and should not show large variation.

The ground resistance factors have been adopted for action combination 1 and 2...The FOS for combination 2 will work out to 2.0 with respect to ultimate carrying capacity.

For seismic combination and accidental combinations depending upon the situation. the partial factor for ground resistance has been suggested.

Table 1 Partial factor for ground resistance for shaft in compression

Component	Symbol	For bored and continuous auger Pile		Driven Piles		GRS for Seismic Combination	GRA for accidental combination		Model Factor for bored and auger pile and driven piles
		GR1	GR2	GR1	GR2		For Vehicle collision, Log Impact and barge collision	Ship Collision	M
For Base resistance	γ_b	1.0	1.7	1.0	1.5	1.40	1.25	1.40	1.2
For shaft resistance	γ_s	1.0	1.4	1.0	1.3	1.15	1.25	1.15	1.2
Total combined resistance	γ_t	1.0	1.7	1.0	1.5	1.40	1.25	1.40	1.2

2.4 Model Factor (to be used in case shear parameters are used for estimation of pile capacity)

Model factor to be used to estimate pile capacity using shear parameter in addition to ground resistance factors.

When the pile capacity is calculated using the ground parameters and resistance factors, in addition.

Model factor to be adopted to arrive at the pile capacity. Model factor has been chosen to achieve the desired factor of safety. Factor of 1.2 has been suggested. The Model factor basically increases the ground resistance factor for not taking into the characteristic property of material. For computation of compressive and tensile resistance of the ground of piles, in vertical direction partial factor on material set value 1 only be used in all combinations as adjustment of capacity for various combinations has been taken care in the ground resistance factors suggested.

So factor of safety for combination 2 is 1.2 x 1.7=2.04 for bored and CFA Piles. Further taking partial factor on action as 1.1 as an average value the FOS will be 2.24. For driven piles the FOS will be 1.2 x 1.5 = 1.8. For seismic combination 25% overstressing is allowed as per present code. Retaining the same concept FOS is 1.4x1.2=1.7. For ship impact same value is retained. For other vehicle collisions 1.25 x1.2 =1.5 and for combination 1 is 1x1.2x1.4= 1.7. So The FOS will be 2.04,1.7 and 1.5.

2.5 Methods of designing piles

The present code covers 5 methods for designing the pile. Same 5 methods are retained and the euro code also covers these 5 methods. The methods are (1) calculation from shear parameters (2) calculation from ground in situ profiles (3) direct load tests (4) dynamic tests (5) pile driving formula.

For the first method the characteristic capacity is to be arrived using the formula and shear parameters and to be divided by ground resistance factor and model factor to arrive at the design resistance as explained in the previous para.

Other methods are capacity from static load testing, ground in situ profiles and dynamic test and using pile driving formula Correlation factors (weight age factors for methods) are to be used instead of model factor when the capacity is computed using field in- situ tests. This correlations factors convert the estiamated/ observed capacity of piles to characteristic capacity.

It is to be noted with the interest that correlation factors differ, for static pile load test, computation from ground profiles and from dynamic methods. The numbers increase in same order. For structures having sufficient strength and stiffness to transfer load from weak to strong piles the values can be divided by 1.1 for piles in compression. Using this recommendation.

(a) From static pile load tests.

Table 2 Correlation factors

Correlation factors				
Number of tests	1	2	3	4
On Mean Value	1.40	1.33	1.29	1.26
On Minimum Value	1.40	1.22	1.11	1.04

The factor safety will be on mean value: for single pile 1.70 x 1.40 = 2.38 and on the min value also same.

For two pile testing: it will be 1.70 x 1.33 = 2.26 or1.70 x 1.22 = 2.07

For 3 pile testing: it will be 1.70 x 1.29 = 2.19 or 1.70 x 1.11 = 1.89

For 2 pile testing the FOS will be come 2.00 (approx.).

(b) For Ground Profile tests:

Table 3 correlation factors

Correlation factors					
Number of tests	1	2	3	4	5
On Mean Value	1.40	1.33	1.29	1.26	1.24
On Minimum Value	1.40	1.26	1.2	1.17	1.14

Up to 3 locations the values are same.

FOS single location testing = 2.38 on mean value and min value

FOS for two locations testing = 1.33 x 1.70 = 2.21 on mean value

On minimum value = 1.26 x 1.70 = 2.14

FOS 3 location testing = 1.29 x 1.70 = 2.19 and 2.04

FOS 4 locations = 1.26 x 1.70 = 2.14 and 1.99

(c) From Dynamic Tests

Table 4 Correlation Factors

Number of tests	≥ 2	≥ 3	≥ 10	≥ 15	≥ 20	Remarks
On Mean Value	1.94	1.85	1.83	1.82	1.81	Where signal matching is done. The factors can be multiplied by 0.85
On Minimum Value	1.90	1.76	1.70	1.67	1.66	

Up to 10 locations the factors will vary from 3.30 to 3.1 on mean values.

and on minimum values = FOS 3.23 to 2.89

(d) Conclusions :

a. The FOS is not uniformly 2

b. The correlation factor Increase from static load test to Dynamic load tests which indicate the reliability of the method.

Minimum 2 or 3 Pile static load test when conducted the FOS will be 2.00 and cannot get away with single pile test For cone penetration test at least at 4 locations the cone penetration test have to be conducted to achieve FOS 2.0.

Dynamic test results to be less reliable .The FOS will be higher varying from 3.30 to 3.1 on mean value and 3.23 to 2.89 on Minimum value.

If Signal matching is done then this will be 2.80 to 2.63 on mean values and on min value 2.74 to 2.45.

The code wants the FOS to be 2.75 for this method.

2.6 Calculation of pile capacity for piles resting on rock and intermediate Geo-Materials

Behavior of pile foundation resting on Rock

The load –penetration curve for rock of medium strength or less (≤ 100 MPa) has a large plastic component despite the brittle nature of rock. The displacements required to mobilize the full bearing capacity of such rocks are very large and that a factor safety of 3 or 4 is required to limit the displacement to less than 2% of the diameter .Very brittle rocks (≥ 150 Mpa) do not exhibit plastic load -penetration curve and once the maximum strength is exceeded at any point in the brittle material total collapse occurs. Hence to limit the displacement, as such large displacements cannot be achieved at the site, the serviceability limit state approach has been suggested as a principal approach for design and verification by load tests. However only the capacity of pile at the ultimate state is required to be verified

The allowable load on the pile under serviceability limit state shall be calculated from any of the following methods

- By calculation using the ground parameters obtained from site investigation
- Directly from static load tests

For method (1) static load tests need to be performed for correlation of pile capacity obtained by calculation. Both Poulos and Davis and M. J . Tomlinson recommend allowable load concept by adopting suitable factor of safety on ultimate strength of rock. Hence in this code this methodology is retained. The ground resistance factors are adjusted to suit the present code which means the present code is converted to limit state approach in this draft. Piles have to be designed only under ULS using ground resistance factor and model factor 1.15.The piles have to be designed under rare combination of loads and load tested at site. The ground resistance factors for piles on rock are as follows.

Table 5 Partial Factor for ground Resistance (γ_r)

Component	Symbol	For bored and continuous augur flight piles		GRS for Seismic	GRA for Accidental Combination	
		GR3	GR4		For Vehicle collision, Log Impact and barge collision	Ship Collision
For Base resistance	γ_s	1.5	2.4	1.9	1.5	1.9
For Socket resistance	γ_s	3.0	4.8	3.9	3.0	3.9
For Combined resistance	γ_t	1.5	2.4	1.9	1.5	1.9

FOS for combination A2 for end bearing 2.4x1.15 =2.76. For socket friction 4.8X1.15= 5.5

FOS for seismic combination and ship accidents for end bearing 1.9x1.15=2.2. For socket friction 3.9X1.15= 4.48

FOS for combination A1 and other accidental combinations for end bearing 1.5x1.15=1.7. For socket friction 3.0X1.15= 3.45

2.7 Tensile resistance of pile foundation

For combinations 1and 2 the partial factors are shown below 2 have been adopted . Also suggests model factor, .Our present code suggests a FOS of 3.0. Hence a model factor of 1.5 has been suggested so that under combination 2. a FOS of 3.0 is achieved (SLS Case).For other combinations the resistance factor has been adjusted in the same principles as suggested earlier for other cases

Table 6 Partial Factor for Resistance for Tension Piles in soil

Resistance	Symbol	GR5 For combination 1	GR6 For combination 2	GRS for Seismic	GRA for Accidental Combination		Modal Factor
					For Vehicle collision, Log Impact and barge collision	Ship Collision	
Shaft in Tension	γ_{st}	1.0	2.0	1.60.	1.25	1.60	1.5

2.8 Routine load test

Routine load test for piles on soil shall be conducted to 1.5 times of rare combination of load and settlement to be restricted to theoretical settlement calculated using shear parameters. For piles seated on rock 1.0 time rare combination of load may be load tested

2.9 Piles subjected to lateral loads

Method suggested by Tomlinson (BROMS method and Brinch Hansen's method) has been recommended. Poulos and Davis also suggest these methods for ULS verification. Use of special software applicable for pile foundations has also been recommended.

For SLS condition using soil springs, Matlock and Reese method or equivalent cantilever method has been recommended.

2.10 Factors for SV loading 2.10.1 Analysis on Partial safety factor for using SV loading on piles foundation

A) **Normal Loading :** Factor of safety (Load 100% Resistance Ultimate)

1) Calculation of Pile FOS using shear parameters

		MF x PrAXGR
Combination 1	DL:LL 50:50 =	1.2 x 1.425x1.0 = 1.71
	DL:LL 70:30 =	1.20 x 1.395x1.0 = 1.67 (Average 1.69)
	Ground Strength alone = 1.20	

		MF x PFAXGR
Combination 2	DL:LL 50:50 =	1.20 x 1.15 x 1.70 = 2.346
	DL:LL 70:50 =	1.20 x 1.09 x 1.70 = 2.22

Ground strength alone 1.2 x 1.7 = 2.04

2) Calculation of piles FOS using ground test results

The factors are some because the modal factor and CR2 factor for 3 locations are same 1.20. GR factor also remain same.

3) Calculation of pile FOS from static load test

Taking 3 static loads conducted

Combination 1 DL: LL 50:50 = 1.11 x 1.425 = 1.580

DL: LL 70:30 = 1.11 x 1.395 = 1.540

Combination 2 DL: LL 50:50 = 1.11 x 1.15 x 1.70 = 2.17

DL: LL 70:30 = 1.11 x 1.09 x 1.70 = 2.05

Ground strength alone 1.90

4) Calculation of pile FOS from Dynamic tests

Taking tests conducted at two locations

Combination 1 DL:LL 50:50 = 1.50 x 1.425 = 2.13

DL:LL 70:30 = 1.50 x 1.395 = 2.09

Combination 2 DL:LL 50:50 = 1.50 x 1.15 x 1.70 = 2.93

DL:LL 70:30 = 1.50x 1.09 x 1.70 = 2.77

B) SV Loading Case

Reduce the ground resistance factor by 15% for combination 2

1) FOS using shear Parameters

Combination 1 DL: LL 50:50 = 1.20 x 1.250 = 1.50 (88%)

DL: LL 70:30 = 1.20 x 1.29 = 1.55 (95%)

On Ground strength alone 1.20

Partial factor on actions will work out to 1.35 minimum . Hence the factor will be above 1.6.

Combination 2 DL:LL 50:50 = 1.20 x 1.00 x 1.70 x0.85 = 1.73 (78%)

DL:LL 70:30 = 1.20 x 1.00 x 1.7 x 0.85 = 1.73 (78%)

2) FOS using ground test result: Value will be same as above

3) FOS using static test result:

Combination 1 DL: LL 50:50 = 1.11 x 1.25 = 1.39 (88%)

DL: LL 70:30 = 1.11 x 1.290 = 1.43 (92%)

Ground Strength alone 1.11

Partial factor on actions will be more than 1.35.. Hence factor will be more than 1.6

Combination 2 DL:LL 50:50 = 1.11 x 1.0 x 1.7 x0. 85 = 1.60 (74%)

DL:LL 70:30 = 1.11 x 1.0 x 1.7 x0. 85 = 1.60 (78%)

Ground strength alone 1.60

4) FOS using Dynamic tests

Combination 1 DL: LL 50:50 = 1.50 x 1.25 = 1.88 (88%)

DL: LL 70:30 = 1.50 x 1.29 = 1.94 (93%)

On Ground Strength 1.50

Combination 2 DL: LL 50:50 = 1.50 x 1.0 x 1.7 x 0.85 = 2.16 (74%)

DL: LL 70:30 = 1.50 x 1.0 x 1.75 x 0.85 = 2.16 (78%)

Ground strength alone 2.16

Factor of safety remains 1.60 and above for normal methods and for dynamic methods it will be 2.20.

Target for normal methods is 2.0 for normal loading: 25% of over stressing when compared to normal load. Hence 15% reduction for combination 2 can be accepted for ground resistance factor.

3. DESIGN OF WELL FOUNDATIONS

3.1 Design approach for Well Foundations

Well foundations are covered using existing provisions of the present code. Hence present IRC:78 has been followed. However the present code is converted into limit state approach so that it is in line with other chapters. Moreover it has to follow the load combinations and partial factors given in **Table B.4** of IRC:6.

3.2 Limit State Verification of side and base Resistance

The side and base resistances of well foundation shall be verified for all the four combinations outlined in **Table B.4** of IRC:6-2017. These are the only two checks for well foundations required

3.3 Loads, Combinations, partial factor material set values and resistance factor

The principles of combination is as follows

combination 1 + Partial factor on material set value 1+ Resistance factor for base and side

combination 2 + Partial factor on material set value 2 + Resistance factor for base and side

Seismic combination + Partial factor on material set value 2 + Resistance factor for base and side

Accidental combination + Partial factor on material set value 2 + Resistance factor for base and side

Meaning of "+" combined with

3.4 Partial factor for Geo-technical Material

Two sets of partial factors are shown in **Table 2** Viz set 1 and set 2.

The factors shown under Set 1 shall be used to check Action Combination 1 and factors shown under set 2 shall be used for remaining combinations.

Table 1 Sensitivity Analysis Table for side resistance

In combination 1 side earth resistance will be almost fully utilized as shown below.

Value	Actual	Reduced	FOS
For φ 25° (kp - ka)	(3.91– 0.35) = 3.56	(3.91 – 0.35×1.5) = 3.39	1.05
Fφ φ = 30° (kp - ka)	(5.74 – 0.30) = 5.44	(5,74 – 0.30×1.5) = 5.29	1.03
For φ = 35°	(9.15– 0.22) = 8.92	(9.15 – 0.22×1.5) = 8.85	1.012

However this will not be governing case. The utilisation will be low in case of action combination 2. When action combination 2 is adopted side resistance factor of FOS is as follows.

Table 2 sensitivity table for side resistance

Actual	Reduced	FOS
For φ 25° ($k_p - k_a$) = (3.91 – 0.35) = 3.56	(2.89 x 0.85 – 0.42 x 1.3) = 1.91	1.86
For φ 30° (5.74 – 0.30) = 5.44	(3.85 x 0.85 – 0.35 x 1.3) = 2.82	1.93
For φ 35° (9.15 – 0.23) = 8.92	(5.4 x 0.85 – 0.29 x 1.3) = 4.21	2.11

The FOS requirement is 2.0 and 1.6 for normal and seismic combinations as per present code. Adopting a side resistance factor of 1.0 for all combinations FOS will be 1.93 for combination 2. For seismic combination and accidental combination FOS will be 1.55

3.5 Partial Factor on ground Resistance for foundations resting on soil

The partial factor for ground resistance is as given below. Partial resistance factor average for actions in combination 1 shall be taken as 1.40. Take ground resistance factor as 1.35

Taking 1.4 as partial factor on loads in combination1 as average, the FOS on base resistance will work to 1.4 x 1.35 = 1.90. Principle is, actions are factored, when partial factor on materials are not applied

For combination 2 reduction in Nq, Nγ. Due to adoption of set 2 value.

Table 3 sensitivity table for base resisatance

φ	Nq	Nγ	φ reduced	Nq	Nγ	Reduction in Nq	Reduction in Nγ
25	10.66	10.88	20.5	6.4	5.39	60%	50%
30	18.40	22.40	25	10.66	10.88	58%	49%
35	33.30	48.05	29	16.8	20.0	50%	42%

Average reduction in Nq = 56% and in Nγ = 47%

As the depth factor dominate taking the bearing capacity reduced by 56% FOS will 1.78.

Hence ground resistance factor can be kept as 1.10. This will ensure FOS of 1.95 for combination 2. For seismic and accidental combination partial factor on ground resistance can be kept as 1.0 as the partial factor for seismic force is 1.5 which includes Hydrodynamic force also. The FOS for seismic and accidental combinations will work out 1.78.

3.6 Summary of Resistance factor

TABLE 4
Summary of resistance factor are shown for all combinations

Combination	Combination (1)	Combination (2)	Seismic Combination	Accidental Combination
Partial Factor to be used On Materials	Set M1	Set M2	Set M2	Set M2
Ground resistance factors	1.35	1.10	1.0	1.0

3.7 Verification of base resistance and base contact area for wells resisting on soil

The base pressure will be verified for all the combinations along with the appropriate partial factor on material and partial factor on resistance. 80% of contact area of base shall be ensured. Maximum base pressure of 2.5 MPa has been limited to

3.8 Verification of Base Resistance and base contact area for wells resting on rock

The base pressure will be verified for all the combinations. The partial factor on materials shall be used for estimating Geo-technical actions and for resistance. The base resistance capacity will be based the crushing strength of rock with resistance factor 4.5. The base contact area shall be ensured a minimum of 80% and 67% for appropriate combinations.

3.9 Serviceability Limit State Verification

The differential settlement (angular distortion) of the foundation will be verified by using Quasi permanent combination and the settlement will be restricted as given in Clause no. 5.1.4. . The contact area should be 100% foundations resting on soil and 80% resting on rock with FOS mentioned to be checked under frequent and rare combination. The allowable pressure on soil and FOS has been stipulated *i.e.* 1.25 MPa and 2

3.10 Structural Design of Foundation

The structural design of foundation shall be carried out for all the combination as per IRC:112

3.11 Factors adjusted for SV loading

3.11.1 *Analysis of partial safety factor on soil for SV loading for base resistance*

A) Normal loading case

For Combination 1 DL:LL 50:50 = Total FOS= 1.425 x 1.35 x 1.0 = 1.92
DL:LL 70:30 = Total FOS =1.395 x 1.35 x 1.0 = 1.88
FOS on Soil = 1.35

For Combination 2 DL:LL 50:50 = Total FOS= 1.15 x 1.1 x 1.78 = 2.25
DL:LL 70:30 = Total FOS = 1.09 x 1.1 x 1.78 = 2.13
FOS on Soil = 1.96

B) With SV Loading Case

For Combination 1 DL:LL 50:50 = Total FOS = 1.25 x 1.20 x 1.0 = 1.50 (78%)
DL:LL 70:30 = Total FOS =1.29 x 1.20 x 1.0 = 1.55 (82%)
FOS on Soil = 1.20

For Combination 2 DL:LL 50:50 = Total FOS = 1.0 x 1.10 x 1.78 = 1.96 (89%)
DL:LL 70:30 = Total FOS = 1.0 x 1.10 x 1.78 = 1.96 (89%)
FOS on Soil = 1.96

Hence Resistance Factor of 1.20 and 1.1 can be taken. For SLS check the FOS shall be reduced to 1.6 against ultimate bearing capacity while checking under rare combination. For Foundation resting on Rock, base Resistance Factor of 4.5 is retained. Reference can be made to open foundation calculation.

INTRODUCTION FOR THE WORKED OUT EXAMPLES

The foundation design presently is being carried out by using Allowable load method. In this procedure the partial factor on Loads is taken as 1.0. The ultimate bearing capacity reduced by a suitable factor of safety leads to allowable bearing capacity based on which the allowable loads are calculated and compared. This procedure was followed by most of the countries in the past. The British Code of Practice on foundation BS 8004 had followed this approach till 2009.

However, in the year 2004, the U.K. as well as other European countries switched over to Limit state of design approach in which two level checks were proposed to check the safety of foundation viz., ultimate limit state and serviceability limit state. The existing practice of allowable load method for designing of foundations was discontinued from 2010 onwards. Between 2004 and 2009 both approaches were retained. In order to be in line with universal approach, the code of Practice of design of concrete bridges, steel bridges and Code of Practice of design of bearings were revised to the Limit State Concept and published by IRC during the past ten years. Hence it became absolutely necessary to revise IRC:78 code of practice for design of foundations to suit the Limit State Concept. IRC:78 is being retained which is based on allowable load method for some more time. This part of IRC:78 Part 2 will stipulate the design procedure for Limit State approach. Beside publishing the Code, it was decided to add an explanatory note to the code and also worked out examples so that the Engineers can understand easily the reasons behind the clauses while carrying out the design calculations.

Basically the earlier concept of factor of safety appears on three forms in this Code. The partial factor both on Geo-technical and structural loads, partial factor on Geo-technical material similar to partial factor on steel and concrete and partial factor on ground resistance. Use of partial factor on Geo-technical material to arrive at the design value from the characteristic value of material is a new concept.

The worked out examples presented here, cover the design of open foundation, pile foundation and pier well foundation resting on both on soil and rock. The design of open foundation covers both cases with and without water table. The pile foundation design covers the conventional method of designing the pile using the shear parameters. The existing method of design of well foundation converted to Limit State approach has also been presented. Simultaneously the same examples have been worked out using the design procedure being followed by the present method. This procedure has been followed to calibrate the Limit State Code. Basically these examples will show the design engineers the following aspects in details:

1) How to combine the loads for different load combinations;
2) How to apply partial factor on structural loads and on Geo-technical loads.

3) How to apply partial factor on Geo-technical material for checking the stability, to compute capacity of the supporting medium and further using the partial factor for ground resistance, on the computed capacity of the supporting medium, to arrive at the design ground resistance.

4) How to satisfy the different requirements of limits prescribed in the Code.

In order to show the amount of saving expected, parallel calculations by the existing, allowable load method also has been presented. It is made very clear that whenever a new concept is adopted, the results should not deviate very much from the current practice. This is specially done in order to infuse confidence. Other countries have followed this approach. Hence similar approach has been adopted here also.

1. Example on Open Foundation by Working Stress Method as per Present IRC:78 and by Limit State Method

1.0 Input Data and Load Calculations

The overall width of the deck is 17.0m (with 16m wide carriage way). The superstructure comprises of three spans continuous (deck continuity only) precast pretensioned girder with cast in-situ RCC deck slab. POT-PTFE bearings under superstructure are supported on RCC rectangular cap which is supported on circular pier. In this example we design the foundation of Anchor pier using Working stress method and limit state method for different cases.

1.1 Salient Data:

Right Span on the pier	=	27.024	m
Left Span on the pier	=	27.025	m
Radius of curvature	=	500	m
Total width of the Bridge	=	17.000	m
Total carriageway & footpath if considered	=	16.000	m
Finished Road Level (FRL)	=	250.424	m (For tallest pier)
Bearing level	=	248.009	m
Pier cap top level	=	247.509	m
Depth of pier cap	=	2.000	m
Pier top level	=	245.509	m
Ground level	=	243.226	m
Earth cushion below ground level	=	1.200	m
Pier bottom level	=	242.026	m
Foundation Level	=	239.726	m
Clear height of Pier	=	3.483	m
FRL to Foundation	=	10.70	m
Avg Expose height for wind load calculations (Plain Terrain)	=	12.00	m
Dia for Circular Pier	=	2.5	m

In case of curved span, the C. G of superstructure is not in line with the centerline alignment therefore eccentricity due to curvature has to be taken into account.

Eccentricity	0.12	0.12	m

(a): Centrifugal Force

Centrifugal force = $W.V^2/127R$

where,
W is the wt. on pier
V is the speed of vehicle
R is the radius of curvature

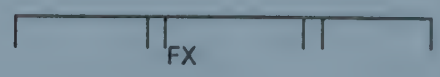
Horizontal force on a pier in normal condition:

Applied horizontal force on both the spans: As per Clause 211.5.2 of IRC:6-2017,

Force on fixed bearing = $F_h - \mu^*(R + L)$ OR $F_h/(1+\Sigma n_{R,L}) + \mu^*(R - L)$

Total horizantal force due to Braking force:

With Class 70RW+2L Class A F_h = 42.44 t



Number of supports to the Right of Fixed Pier 3

Number of support to the Left of Fixed Pier 1

	Reaction to the Right of Fixed Bearing	Reaction to the Left of Fixed Bearing
Dead Load	1155.00	385.000
SIDL	67.50	22.50
Surfacing	135.00	45.00
Live Load*	337.28	0.00
Coeff of Friction (μ)	0.05	0.03
Frictional Force	84.74	13.58

*{4 lane of class A (=(55.4*1+50)*4*.8) Considered conservatively}

μL	=	13.58 t	
μR	=	84.74 t	With Live load case
μR	=	67.88 t	Without Live load Case
n_R	=	2	
n_L	=	1	
Horizontal force on a bearing	=	92.38 t	With Live load case *(on conservative side we take min of ηR & ηL)*

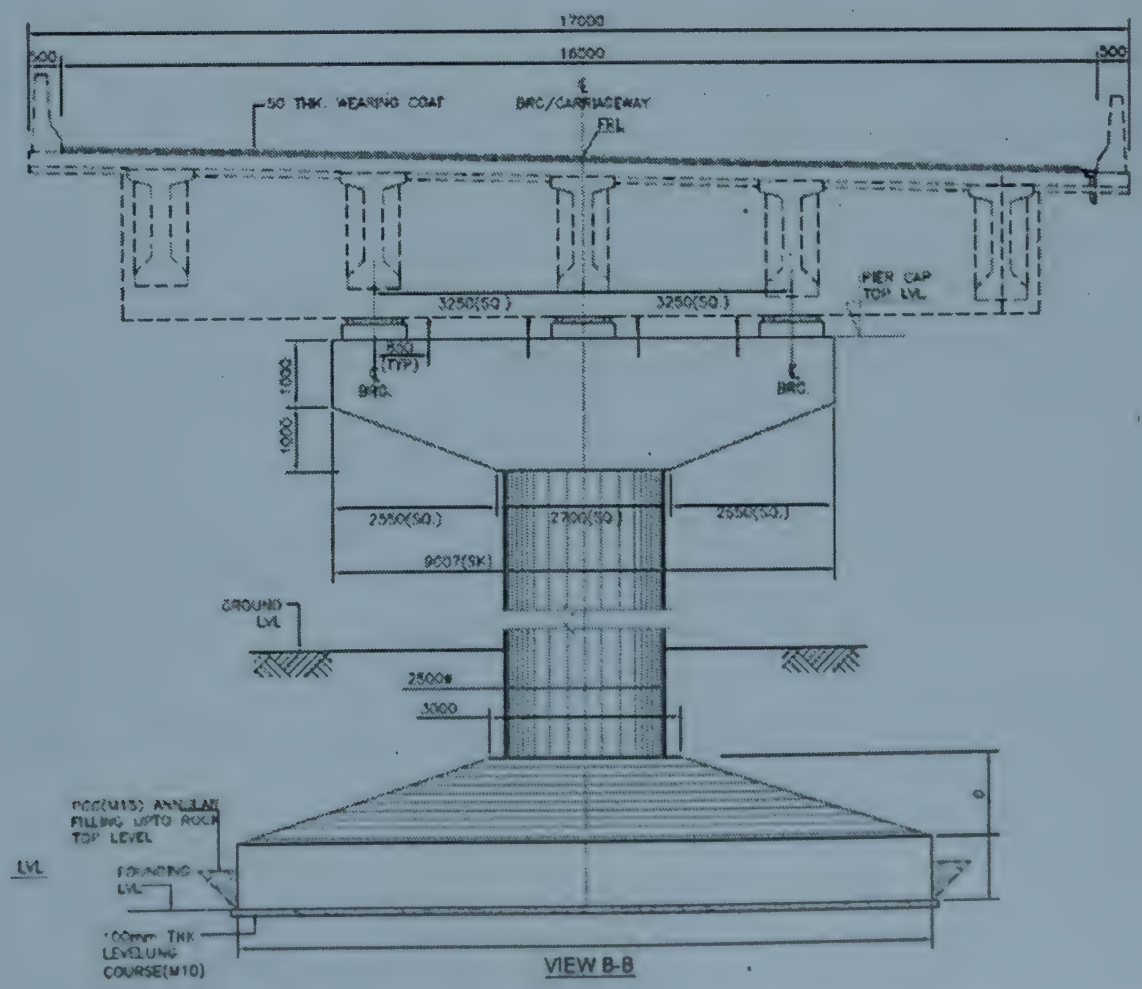


Fig. 1 Typical Cross section in Transverse Direction

b: Buoyant force due to pier & foundation:

(Typical calculations)

As per Clause 213 of IRC:6-2017,

Weight of foundation	=	885.5	t
Weight of Soil	=	357.8	t
Weight of Pier up to GL	=	5.9	t
Density of water	=	1.00	t/m^3

Assuming Water level is at Ground level.

Buoyant force due to foundation	=	354.2	t
Buoyant force due to Soil	=	178.9	t
Buoyant force due to Pier	=	5.9	t
Buoyant force due to Pier Cap	=	0.0	t
Therefore, total buoyant force	=	539.0	t

1.2 CALCULATION OF LOAD

Length of Foundation (in longitudinal Dir.) L	=	14.00	m
Width of Foundation (in Transverse Dir.) B	=	11.00	m
Depth of Footing	=	2.30	m
Gross area of footing	=	154	m^2
Section modulus in longitudinal direction, Z_L	=	359.33	m^3
Section modulus in transverse direction, Z_T	=	282.33	m^3
Therefore, total buoyant force	=	539.0	t

1.2.i Loads from superstructure:

Type of load	Right Span	Left Span	Long. Moment	Trans. Moment
	(t)	(t)	(tm)	(tm)
Dead load	385.00	385.00	0.00	93.74
SIDL	22.50	22.50	0.00	5.48
Surfacing	45.00	45.00	0.00	10.96
FPLL	0.00	0.00	0.00	0.00
LL (Max. Vcase)	65.34	83.04	96.00	349.71

1.2.ii Loads from substructure:

Weight of pier cap	=	140.00	t
Weight of pier	=	42.92	t
Depth of earth cushion	=	1.20	m
Density of cushion	=	2.00	t/m^3
Weight of earth cushion	=	357.77	t

Weight of Foundation	=	885.50	t
Weight of Foundation & cushion	=	1243.27	t
C.G. of Footing & Cushion from Foundation Levels	=	1.7	m

2.iii) CALCULATION OF SEISMIC FORCES-

CALCULATION OF SEISMIC COEFFICIENT

SEISMIC ZONE = III

LONGITUDINAL SEISMIC COEFFICIENT

Zone factor for seismic zone III, Z	=	0.16		
Sa/g	=	2.500		
Importance Factor, I	=	1.2		
Response Reduction Factor, R	=	3.00		
A_h = (Z/2) x (I/R) x (Sa/g)	=	0.080	**0.080**	**say**

TRANSVERSE SEISMIC COEFFICIENT

A_h = (Z/2) x (I/R) x (Sa/g)	=	0.080	**0.080**	**say**
VERTICAL SEISMIC COEFFICIENT	=	0.000	**0.000**	**say**

HORIZONTAL FORCES (BRAKING AND BEARING FRICTION)

** For seismic load calculations, consider only 20% LL* 0.20

With POT-PTFE

	Reaction to the Right of Fixed Bearing	Reaction to the Left of Fixed Bearing
Dead Load	1155.00	385.00
SIDL	67.50	22.50
Surfacing	135.00	45.00
Live Load	84.32	0.00
Coeff of Friction (μ)	0.05	0.03
Frictional Force	72.09	13.58

The structure under the fixed bearing shall be designed to withstand the full seismic and design braking/tractive force.

Seismic Longitudinal Case

With POT-PTFE

Braking force		8.49	t
No. of spans fixed to the pier		3	
Seismic force on superstructure		217.20	t
Total horizontal force on superstructure		225.69	t
Frictional force on fixed bearing	$F_h/(1+\Sigma n_{R,L}) + \mu^*(R - L)$	171.36	t
or	F_h	225.69	
Thus, max force on a pier		225.69	t

IN LONGITUDINAL DIRECTION

LONGITUDINAL SESMIC FORCE DUE TO	Force (t)	Lever arm (m)	Moment (tm)
Superstructure	225.69	8.28	1869.39
Pier cap	11.20	6.91	77.43
Pier	3.43	4.04	13.88
Footing & Cushion	99.46	1.65	164.47
Total longitudinal Seismic force	**339.78**		**2125.17**

IN TRANSVERSE DIRECTION

TRANSVERSE SESMIC FORCE DUE TO	Force (t)	Lever arm (m)	Moment (tm)
Live Load (Horizontal) at 1.2 above FRL	2.37	11.90	28.25
Dead Load (Right span)	30.80	10.06	309.77
Dead Load (Left span)	30.80	10.06	309.77
Surfacing	7.20	10.70	77.03
SIDL	3.60	11.20	40.31
Pier Cap	11.20	6.91	77.43
Pier	3.43	4.04	13.88
Footing	99.46	1.65	164.47
Total transverse Seismic force	**188.87**		**1020.90**

SUMMARY OF SEISMIC FORCES & ITS MOMENT		
	Force (t)	**Moment (tm)**
Transverse Direction	188.87	1020.90
Longitudinal Direction	339.78	2125.17
Vertical Direction	0.00	-

1.2.iv CALCULATION OF FORCES DUE TO WIND - WITH LIVE LOAD

I WIND FORCE ON SUPERSTRUCTURE (As per Clause 209.3 of IRC:6-2017)

Basic wind speed (V_b) (As per Fig.10)	41.89 m/s
H (avg. ht. of exposed surface from the ground level)	12.00 m
From table 12	
V_z (hourly mean speed of wind at height H)	28.36 m/s
P_z (horizontal wind pressure at height H)	483.22 N/m²
Since table 12 is for basic wind speed (V_b) of 33 m/s	
So, for V_b of 41.89 m/s 0m/s	
V_z calculated	36.0 m/s
P_z calculated	778.6 N/m²
In hilly areas due to funneling, P_z is increased by 20%	778.6 N/m²

As per Clause 209.3.3

Transverse wind force F_T (in N) = $P_z \times A_1 \times G \times C_D$ **eq. 1**

Here,

A_1 is the area of the superstructure as seen in elevation including the floor system and railing

Height of left 27.025m span including height of crash barrier, d	=	3.500	m
Height of right 27.024m span including height of crash barrier, d	=	3.500	m
So, Area A1 at which F_T is acting (left 27.025m Span)	=	3.500	m²/m
So, Area A1 at which F_T is acting (right 27.024m Span)	=	3.500	m²/m
G is the gust factor (spans up to 150 m)	=	2	
C_D is the drag coefficient			
width of deck (b)	=	17.0	m
b/d	=	4.86	m
For bridge decks supported by single beam, C_D (For above b/d ratio)	=	1.36	
For bridge deck supported by two or more beams, C_D	=	2.04	(1.5 times single beam)

Putting all these values in eq.1

F_T calculated	27.025m Span	=	11095.639	N/m
	27.024m Span	=	11095.639	N/m
F_T (For left 27.025 m span)		=	30.57	t
F_T (For right 27.024 m span)		=	30.57	t
Long. Force on superstructure F_L (25% of F_T) (As per Clause 209.3.4) (1 ,27.025 m span)		=	7.64	t
Long. Force on superstructure F_L (25% of F_T) (As per Clause 209.3.4) (2,27.024 m span)			15.28	t

Upward or downward vertical wind load F_V (in N) (As per Clause 209.3.5)	=	$P_Z \times A_3 \times G \times C_L$	**eq.2**
Here,			
So, Area A_3 at which F_T is acting (left 27.025m Span)	=	17.0	m^2/m
So, Area A_3 at which F_T is acting (right 27.024m Span)	=	17.0	m^2/m
Width of deck	=	17.00	m
C_L is the lift coefficient	=	0.75	
Other parameters P_Z and G remains same as used above			
Putting all these values in eq.2,			

F_V(+/-)	left 27.025mSpan	=	19855.354	N/m
	right 27.024m Span	=	19855.354	N/m
F_V (For left 27.025 m span)		=	54.70	t
F_V (For right 27.024 m span)		=	54.70	t

II WIND FORCE ON LIVE LOAD

Since hourly mean speed of wind (V_Z) is 36m/s, so wind forces need to be considered on live load (As per Clause 209.3.7)			
Transverse wind force F_T (in N) (acting at a ht. of 3m above roadway)	=	$P_Z x A_3 x G x C_D$	**eq.3**
Height of crash barrier above roadway	=	1.10	m
Net ht. at which F_T is acting	=	1.90	m
So Area (L x ht. at which F_T is acting)	=	1.90	m^2/m
Drag coefficient, C_D	=	1.20	(As per Clause 209.3.6)
Putting these values in eq.3, F_T calculated	=	3550.60	N/m
	=	0.36	t/m
F_T (For 27.025 m span)	=	9.78	t
F_T (For 27.024 m span)	=	9.78	t
Long. Force on superstructure FL (25% of F_T) (As per Clause 209.3.4) (1, Left 27.025m span)	=	2.45	t
Long. Force on superstructure FL (25% of F_T) (As per Clause 209.3.4) (2, Right 27.024m span)	=	4.89	t

IN LONGITUDINAL DIRECTION

With POT-PTFE

Braking force	=	42.44	t
Wind force on superstructure	=	22.92	t
Wind force on LL	=	7.34	t
Total long. Force	=	72.70	t

Frictional force on fixed bearing	$F_h - \mu^*(R + L)$	=	-25.61 t
or	$F_h/(1+\Sigma n_{R,L}) + \mu^*(R-L)$	=	107.51 t
or	F_h	=	72.70 t
Total force on a pier			**107.51 t**
Thus, max force on a pier		=	**107.51 t**

LONGITUDINAL WIND FORCE	Force (t)	Lever arm (m)	Moment (tm)
Horizontal Force	107.51	8.28	890.55
on Pier cap	0.50	6.78	3.36
on Pier	0.21	4.04	0.84
Total longitudinal Wind force	**108.22**		**894.75**

IN TRANSVERSE DIRECTION

TRANSVERSE WIND FORCE DUE TO	Force (t)	Lever arm (m)	Moment (tm)
Live Load	9.78	12.75	124.69
Superstructure (27.025 m span)	15.28	10.05	153.57
Superstructure (27.024 m span)	15.28	10.05	153.56
Pier cap	1.98	6.78	13.438
Pier	0.83	4.04	3.352
Total transverse Wind force	**43.16**		**448.61**

SUMMARY OF WIND FORCES & ITS MOMENT

	Force (t)	Moment (tm)
Transverse Direction	43.16	448.61
Longitudinal Direction	108.22	894.75
Vertical Direction (+/-)	54.70	-

2.0 Prob 1 - Foundation Resting on Soil with Water Table at Ground Level

(A_h & A_t = (Z/2) x (I/R) x (Sa/g) = 0.08)

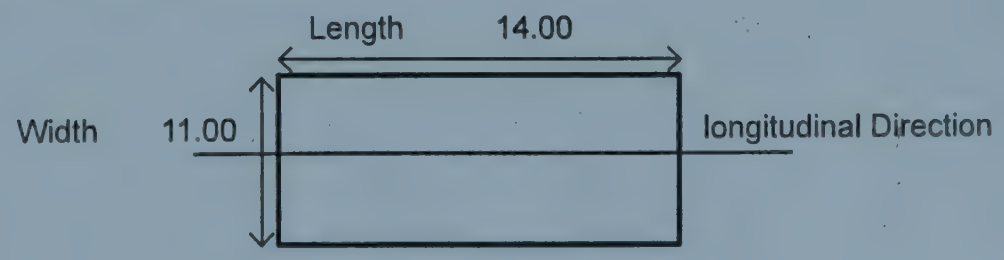


Fig. 2 Plan of Footing

Area of foundation (A_{gross})	=	154.00 m^2
Length of Foundation (in longitudinal Dir.) L	=	14.00 m
Width of Foundation (in Transverse Dir.) B	=	11.00 m
Section modulus in longitudinal direction, Z_L	=	359.33 m^3
Section modulus in transverse direction, Z_T	=	282.33 m^3

Summary of Forces of structure without any factor-

For forces refer input data and load calculations (Buoyant & seismic force and footing wt. can be changed depending upon size of footing)

S.No.	Particular	V	H_T	H_L	M_T	M_L
		(t)	(t)	(t)	(tm)	(tm)
1	DEAD LOAD	770.00	-	-	93.74	0.00
2	Surfacing	90.00	-	-	10.96	0.00
3	SIDL	45.00	-	-	5.48	0.00
4	FPLL	0.00	-	-	0.00	0.00
5	LL (Max. V case)	148.38	9.87	-	349.71	96.00
6	Pier cap	140.00	-	-	-	-
7	Pier	42.92	-	-	-	-
8	Footing & cushion	1243.27	-	-	-	-
9	Buoyant force	539.00	-	-	-	-
10(a)	Horizontal force Basic (Live Load case)	-	-	92.38	-	765.22
10(b)	Horizontal force Basic (No Live load case)	-	-	54.30	-	449.77
10(c)	Seismic Force	0.00	188.87	339.78	1020.90	2125.17
10(d)	Wind Force with LL Case	54.70	43.16	108.22	448.61	894.75

(While Calculating factored force in seismic longitudinal direction, 30% of seismic vertical and seismic transverse force are also considered.)

2.1 Geo-Technical Design of Foundations using Working Stress Method-

σ_v *(t/m²) = Bearing pressure from trapezoidal pr. Distributions in WSM*

Max. Base Pressure $(P/A+M_L/Z_L+M_T/Z_T)$

Min. Base Pressure $(P/A-M_L/Z_L-M_T/Z_T)$

Partial Safety factors

	As Per IRC: 78		
	LC-1	LC-2	LC-3
DL, SIDL	1.00	1.00	1.00
Surfacing	1.00	1.00	1.00
LL & FPLL Load	1.00	1.00	0.20
Wind	0.00	1.00	0.00
Seismic	0.00	0.00	1.00
Bouyancy	1.00	1.00	1.00

i) ***Basic load combination (LC-1)-***

	LC-1
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	1.00
Wind	-
Seismic	-
Bouyancy	1.00

						σ_v (t/m²)	
	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min
Factored load	1941	10	92	460	861	16.6	8.6

ii) ***Basic combination with Wind (LC-2)-***

	LC-2
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	1.00
Wind	1.00
Seismic	-
Bouyancy	1.00

							σ_v (t/m²)	
		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min
Factored load	+	1995	53	108	908	991	18.9	7.0
	-	1886	53	108	908	991	18.2	6.3

+ means Wind downward & - means Wind Upward

iii) ***Seismic Combination (LC-3)-***

	LC-2
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	0.20
Wind	-
*Seismic	1.35
Buoyancy	1.00

**Seismic force are multiply by 1.35 as per Clause 219.8 of IRC: 6-2017*

					σ_v (t/m²)	
V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min
1822	78	459	594	2888	22.0	1.7

Factored load

Summary of bearing pressure from WSM-

i) Min Pressure	=	1.69	t/m²	>0 , Full contact of base, Safe
ii) Max base Pressure				*(Cl 706.3.3.1 of IRC:78-2014)*
Basic	=	16.6	t/m²	≤ 48.52 /2.5 = 19.408 t/sq-m, Safe
Wind	=	18.9	t/m²	≤ 48.52* 1.25/2.5 = 24.26 t/sq-m, Safe
Seismic	=	22.0	t/m²	≤ 48.52* 1.25/2.5 = 24.26 t/sq-m, Safe
Ultimate bearing capacity	=	48.52	t/m²	

2.2 Geo-Technical Design of Foundations using Limit State-

Partial Safety factors

	Table B.4 (IRC: 6-2017)					Table B.3 (IRC: 6-2017)				
	LC-1	LC-2	LC-3	LC-4	LC-5A, 5B	LC-6	LC-7	LC-8	LC-9	LC-10
	Combination-1		Combination-2		Seismic	Frequent		Rare		Quasi
	LL Lead	Wind Lead	LL Lead	Wind Lead	-	LL Lead	Wind Lead	LL Lead	Wind Lead	-
DL, SIDL	1.35	1.35	1.00	1.00	1.35,1	1.00	1.00	1.00	1.00	1.00
Surfacing	1.75	1.75	1.00	1.00	1.75,1	1.20	1.20	1.20	1.20	1.20
LL & FPLL Load	1.50	1.15	1.30	1.00	0.20	0.75	0.20	1.00	0.75	0.00
Wind	0.90	1.50	0.80	1.30	-	0.50	0.60	0.60	1.00	0.00
Seismic	-	-	-	-	1.50	-	-	-	-	-
Buoyancy	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

2.2.1 Serviceability Limit State -

(As per Clause 3.4 Verification of serviceability Limit State)-

σ_v (t/m²) = Bearing pressure from trapezoidal pr. Distributions for SLS check-

i) Check for Loss of contact-

Frequent Combination	LC-6	LC-7
	Combination-1	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.20	1.20
LL & FPLL Load	0.75	0.20
Wind	0.50	0.60
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	σ_v (t/m²) Max	σ_v (t/m²) Min	A_{eff} (m²)
LC-6	+	1949	29	81	599	743	16.8	8.5	154.0
	-	1894	29	81	599	743	16.5	8.1	154.0
LC-7	+	1873	28	65	451	556	15.3	9.0	154.0
	-	1807	28	65	451	556	14.9	8.6	154.0

+ means Wind downward & - means Wind Upward

Area of foundation (A_{gross}) = 154.0 m²

Min % contact area = A_{eff} / A_{Gross}

= **100.0%** **≥ 90 % Safe** *(As per Limit State Code)*

ii) Check for Base Pressure-

Rare Combination

	LC-8	LC-9
	Combination-2	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.20	1.20
LL & FPLL Load	1.00	0.75
Wind	0.60	1.00
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	σ_v (t/m²) Max	σ_v (t/m²) Min
LC-8	+	1991	36	108	731	991	18.3	7.6
	-	1926	36	108	731	991	17.9	7.2
LC-9	+	1976	51	108	823	967	18.4	7.2
	-	1867	51	108	823	967	17.7	6.5

+ means Wind downward & - means Wind Upward

Area of foundation (A_{gross}) = 154.0 m²

Z_L of footing ($L^2B/6$) = 359.3 m³

Z_T of footing ($B^2L/6$) = 282.3 m³

Max. Base Pressure ($P/A+M_L/Z_L+M_T/Z_T$) = 18.4 t/m²

Min. Base Pressure ($P/A-M_L/Z_L-M_T/Z_T$) = 6.5 t/m² ≤ 21.56 t/sq-m, Safe (As per Limit State Code)

Max base pressure is = 18.4 t/m²

Ultimate bearing capacity = 48.5 t/m²

Design Ultimate bearing capacity (48.52/2.25) = 21.6 t/m²

iii) Check for Settlement-

	LC-10
	Quasi
	-
DL, SIDL	1.00
Surfacing	1.20
LL & FPLL Load	0.00
Wind	0.00
Seismic	-
Bouyancy	1.00

Factored load

					σ_v (t/m²)	
V (t)	**HT (t)**	**HL (t)**	**MT (t-m)**	**ML (t-m)**	**Max**	**Min**
1810	0	54	112	450	13.4	10.1

Area of foundation = 154.0 m^2

Z_L of footing (L2B/6) = 359.3 m^3

Z_T of footing (B2L/6) = 282.3 m^3

Max. Base Pressure ($P/A+M_L/Z_L+M_T/Z_T$) = 13.4 t/m^2

Min. Base Pressure ($P/A-M_L/Z_L-M_T/Z_T$) = 10.1 t/m^2

Settlement to be calculated using = **13.4** t/m^2

2.2.2 Ultimate limit state -

σ_v *(t/m²) = Bearing pressure from uniform pr. Distributions in ULS Check.*

A_{eff} = Effective area of contact $\{(L-2e_L)*(B-2e_B)\}$.

A) Loss of equilibrium-

Overturning, Overall stability and sliding of foundation are carried out under in this clause.

Generally for pier this check is not required.

B) As per Clause 3.2 Verification for Bearing Resistance-

i) Load Combination 1

	LC-1	LC-2
	Combination-1	
	LL Lead	Wind Lead
DL, SIDL	1.35	1.35
Surfacing	1.75	1.75
LL & FPLL Load	1.50	1.15
Wind	0.90	1.50
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	e_L (m)	e_T (m)	A_{eff} (m^2)	% A_{eff}	σ_v (t/m^2)
LC-1	+	2916	54	162	1081	1486	0.510	0.371	133.2	86.5%	21.90
	-	2817	54	162	1081	1486	0.527	0.384	132.5	86.0%	21.27
LC-2	+	2897	76	162	1228	1453	0.501	0.424	131.9	85.7%	21.95
	-	2733	76	162	1228	1453	0.532	0.449	130.7	84.9%	20.91

+ means Wind downward & - means Wind Upward

Max. Base Pressure (P/A_{eff}) = **22.0** t/m^2 **≤ 36.12 t/sq-m, Safe**

Min % Effective Contact Area = **84.86** % **≥ 50 % Safe**

(As per Limit State Code)

Characteristic Ultimate bearing capacity = 46.95 t/m^2

Design Ultimate bearing capacity (46.95 /1.3) = 36.12 t/m^2

ii) Load Combination 2

	LC-3	LC-4
	Combination-2	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.00	1.00
LL & FPLL Load	1.30	1.00
Wind	0.80	1.30
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	e_L (m)	e_T (m)	A_{eff} (m^2)	% A_{eff}	σ_v (t/m^2)
LC-3	+	2029	47	141	924	1288	0.635	0.455	128.4	83.4%	15.80
	-	1941	47	141	924	1288	0.663	0.476	127.3	82.7%	15.24
LC-4	+	2012	66	141	1043	1259	0.626	0.519	127.0	82.5%	15.84
	-	1869	66	141	1043	1259	0.674	0.558	125.1	81.2%	14.95

+ means Wind downward & - means Wind Upward

Max. Base Pressure (P/A_{eff})	=	**15.8** t/m^2	≤ 22.97 t/sq-m, Safe
Min % Effective Contact Area	=	**81.21** %	50 %, Safe
			(As per Limit State Code)
Characteristic Ultimate bearing capacity	=	29.86 t/m^2	
Design Ultimate bearing capacity (29.86 /1.3)	=	22.97 t/m^2	

iii) Seismic Combination-

	LC-5A	LC-5B
	Seismic	
	DL Adding	DL Rev.
DL, SIDL	1.35	1.00
Surfacing	1.75	1.00
LL & FPLL Load	0.20	0.20
Wind	-	-
Seismic	1.50	1.50
Buoyancy	1.00	1.00

Factored load	V' (t)	HT' (t)	HL' t)		MT' (t-m)	ML' (t-m)	
	-	-	*Seismic*	*non Seismic*	-	*Seismic*	*non Seismic*
LC-5A	2674	87	510	0	682	3188	19
LC-5B	1822	87	510	0	640	3188	19

As the plastic moment is dependent on sectional capacity. It is assumed that section is designed for the exact requirement. Hence the seismic moment and shear at the base is multiplied by over strength factor to simplify the process.

Plastic hinge can form either in longitudinal direction or transverse direction. In this case we assume that plastic hinge is form in longitudinal direction. So, over strength Factors not apply on V, HT & MT.

Factored load after applying Overstrength Factor of 1.35 in HL & ML -

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	e_L (m)	e_T (m)	A_{eff} (m^2)	% A_{eff}	σ_v (t/m^2)
LC-5A		2674	87	688	682	4329	1.619	0.255	112.9	73.3%	23.69
LC-5B		1822	87	688	640	4329	2.376	0.351	95.2	61.8%	19.13

(HL & ML) = (HL' & ML') 1.35*

(V, HT & MT) = (V', HT' & MT')

LC-5A Max. Base Pressure (P/A_{eff}) = **23.7** t/m^2 ≤ 26.93 t/sq-m, Safe

LC-5B Max. Base Pressure (P/A_{eff}) = **19.1** t/m^2 ≤ 26.15 t/sq-m, Safe

Min % Effective Contact Area = 61.84 % ≥50 % Safe
(As per Limit State Code)

	LC-5A	LC-5B	
Characteristic Ultimate bearing capacity =	29.62	28.77	t/m^2
Design Ultimate bearing capacity (29.62 /1.1) & (28.77 /1.1) =	26.93	26.15	t/m^2

2.3 Summary of WSM & LSM-

Method		Description of load case	Loss of contact			Base pressure Check		
			Contact area of base	Allowable contact area of base	Status	Maximum base pressure (t/m^2)	Design ultimate bearing capacity (t/m^2)	Status
Working State Method		Basic	100%	100%	Safe	16.63	19.41	Safe
		Wind				18.93	24.26	Safe
		Seismic				21.97	24.26	Safe
Limit State Method-	SLS	Frequent	100%	90%	Safe	-	-	-
		Rare	-	-	-	18.44	21.56	Safe
	ULS	Load combination-1	85%	50%	Safe	21.95	36.12	Safe
		Load combination-2	81%	50%	Safe	15.84	22.97	Safe
		Seismic Combination	62%	50%	Safe	23.69	26.93	Safe

3.0 Prob 2 - Foundation Resting on Soil with no Water Table

(A_h & A_t = (Z/2) x (I/R) x (Sa/g) = 0.08)

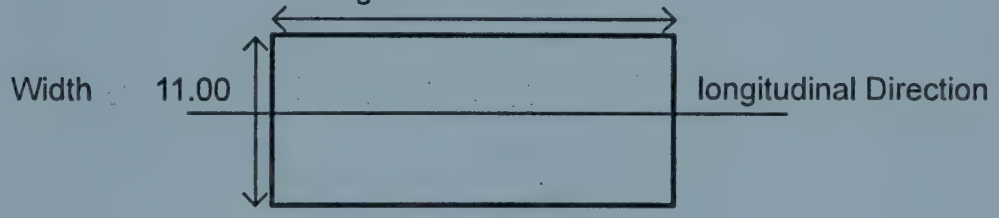


Fig. 3 Plan of Footing

Area of foundation (A_{gross})	=	154.00	m^2
Length of Foundation (in longitudinal Dir.) L	=	14.00	m
Width of Foundation (in Transverse Dir.) B	=	11.00	m
Section modulus in longitudinal direction, Z_L	=	359.33	m^3
Section modulus in transverse direction, Z_T	=	282.33	m^3

S. No.	Particular	V	H_T	H_L	M_T	M_L
		(t)	(t)	(t)	(tm)	(tm)
1	DEAD LOAD	770.00	-	-	93.74	0.00
2	Surfacing	90.00	-	-	10.96	0.00
3	SIDL	45.00	-	-	5.48	0.00
4	FPLL	0.00	-	-	0.00	0.00
5	LL (Max. V case)	148.38	9.87	-	349.71	96.00

S. No.	Particular	V	H_T	H_L	M_T	M_L
		(t)	(t)	(t)	(tm)	(tm)
6	Pier cap	140.00	-	-	-	-
7	Pier	42.92	-	-	-	-
8	Footing & cushion	1243.27	-	-	-	-
9	Buoyant force	0.00	-	-	-	-
10 (a)	Horizontal force Basic (Live Load case)	-	-	92.38	-	765.22
10 (b)	Horizontal force Basic (No Live load case)	-	-	54.30	-	449.77
10 (c)	Seismic Force	0.00	188.87	339.78	1020.90	2125.17
10 (d)	Wind Force with LL Case	54.70	43.16	108.22	448.61	894.75

(While Calculating factored force in seismic longitudinal direction, 30% of seismic vertical and seismic transverse force are also considered.)

3.1 Geo-Technical Design of Foundations using Working Stress Method-

σ_v *(t/m²) = Bearing pressure from trapezoidal pr. Distributions in WSM*

Max. Base Pressure $(P/A+M_L/Z_L+M_T/Z_T)$

Min. Base Pressure $(P/A-M_L/Z_L-M_T/Z_T)$

Partial Safety factors

	As Per IRC: 78		
	LC-1	LC-2	LC-3
DL, SIDL	1.00	1.00	1.00
Surfacing	1.00	1.00	1.00
LL & FPLL Load	1.00	1.00	0.20
Wind	0.00	1.00	0.00
Seismic	0.00	0.00	1.00
Buoyancy	1.00	1.00	1.00

i) Basic load combination (LC-1)-

	LC-1
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	1.00
Wind	-
Seismic	-
Buoyancy	1.00

						σ_v (t/m²)	
	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min
Factored load	2480	10	92	460	861	20.1	12.1

ii) Basic combination with Wind (LC-2)-

	LC-2
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	1.00
Wind	1.00
Seismic	-
Buoyancy	1.00

							σ_v (t/m²)	
		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min
Factored load	+	2534	53	108	908	991	22.4	10.5
	-	2425	53	108	908	991	21.7	9.8

+ means Wind downward & - means Wind Upward

iii) Seismic Combination (LC-3)-

	LC-2
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	0.20
Wind	-
*Seismic	1.35
Buoyancy	1.00

**Seismic force are multiply by 1.35 as per Clause 219.8 of IRC: 6-2017*

						σ_v (t/m²)	
	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min
Factored load	2361	78	459	594	2888	25.5	5.2

Summary of bearing pressure from WSM-

i) Min Pressure = 5.19 t/m² >0, Full contact of base, Safe

ii) Max base Pressure *(Cl 706.3.3.1 of IRC:78-2014)*

Basic = 20.1 t/m² ≤ 97.04 /2.5 = 38.816 t/sq-m, Safe

Wind = 22.4 t/m² ≤ 97.04* 1.25/2.5 = 48.52 t/sq-m, Safe

Seismic = 25.5 t/m² ≤ 97.04* 1.25/2.5 = 48.52 t/sq-m, Safe

Ultimate bearing capacity = 97.04 t/m²

3.2 Geo-Technical Design of Foundations using Limit State-

Partial Safety factors

	Table B.4 (IRC: 6-2017)					Table B.3 (IRC: 6-2017)				
	LC-1	LC-2	LC-3	LC-4	LC-5A, 5B	LC-6	LC-7	LC-8	LC-9	LC-10
	Combination-1		Combination-2		Seismic	Frequent		Rare		Quasi
	LL Lead	Wind Lead	LL Lead	Wind Lead	-	LL Lead	Wind Lead	LL Lead	Wind Lead	-
DL, SIDL	1.35	1.35	1.00	1.00	1.35,1	1.00	1.00	1.00	1.00	1.00
Surfacing	1.75	1.75	1.00	1.00	1.75,1	1.20	1.20	1.20	1.20	1.20
LL & FPLL Load	1.50	1.15	1.30	1.00	0.20	0.75	0.20	1.00	0.75	0.00
Wind	0.90	1.50	0.80	1.30	-	0.50	0.60	0.60	1.00	0.00
Seismic	-	-	-	-	1.50	-	-	-	-	-
Buoyancy	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

3.2.1 *Serviceability Limit State -*

(As per Clause 3.4 Verification of serviceability Limit State)-

σ_v *(t/m²) = Bearing pressure from trapezoidal pr. Distributions for SLS check-*

i) Check for Loss of contact-

Frequent Combination	LC-6	LC-7
	Combination-1	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.20	1.20
LL & FPLL Load	0.75	0.20
Wind	0.50	0.60
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	σ_v (t/m²) Max	σ_v (t/m²) Min	A_{eff} (m²)
LC-6	+	2488	29	81	599	743	20.3	12.0	154.0
	-	2433	29	81	599	743	20.0	11.6	154.0
LC-7	+	2412	28	65	451	556	18.8	12.5	154.0
	-	2346	28	65	451	556	18.4	12.1	154.0

+ means Wind downward & - means Wind Upward

Area of foundation (A_{gross}) = 154.0 m²

Min % contact area = A_{eff} / A_{Gross}

= **100.0%** ≥ 90 % Safe *(As per Limit State Code)*

ii) Check for Base Pressure-

Rare Combination	LC-8	LC-9
	Combination-2	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.20	1.20
LL & FPLL Load	1.00	0.75
Wind	0.60	1.00
Seismic	-	-
Bouyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	σ_v (t/m²) Max	σ_v (t/m²) Min
LC-8	+	2530	36	108	731	991	21.8	11.1
	-	2465	36	108	731	991	21.4	10.7
LC-9	+	2515	51	108	823	967	21.9	10.7
	-	2406	51	108	823	967	21.2	10.0

+ means Wind downward & - means Wind Upward

Area of foundation (A_{gross}) = 154.0 m²
Z_L of footing ($L^2B/6$) = 359.3 m³
Z_T of footing ($B^2L/6$) = 282.3 m³
Max. Base Pressure ($P/A+M_L/Z_L+M_T/Z_T$) = 21.9 t/m²
Min. Base Pressure ($P/A-M_L/Z_L-M_T/Z_T$) = 10.0 t/m²

Max base pressure is = 21.9 t/m² ≤ 43.13 t/sq-m, Safe
(As per Limit State Code)

Ultimate bearing capacity = 97.04 t/m²
Design Ultimate bearing capacity (97.04 /2.25) = 43.1 t/m²

iii) Check for Settlement-

	LC-10
	Quasi
	-
DL, SIDL	1.00
Surfacing	1.20
LL & FPLL Load	0.00
Wind	0.00
Seismic	-
Buoyancy	1.00

						σ_v (t/m²)	
Factored load	**V (t)**	**HT (t)**	**HL (t)**	**MT (t-m)**	**ML (t-m)**	**Max**	**Min**
	2349	0	54	112	450	16.9	13.6

Area of foundation = 154.0 m²
Z_L of footing ($L^2B/6$) = 359.3 m³
Z_T of footing ($B^2L/6$) = 282.3 m³
Max. Base Pressure ($P/A+M_L/Z_L+M_T/Z_T$) = 16.9 t/m²
Min. Base Pressure ($P/A-M_L/Z_L-M_T/Z_T$) = 13.6 t/m²

Settlement to be calculated using = **16.9** t/m²

3.2.2 *Ultimate limit state -*

σ_v *(t/m²) = Bearing pressure from uniform pr. Distributions in ULS Check.*

A_{eff} *= Effective area of contact* $\{(L-2e_L)*(B-2e_B)\}$.

A) Loss of equilibrium-

Overturning, Overall stability and sliding of foundation are carried out under in this Clause. Generally, for pier this check is not required.

B) As per Clause 3.2 Verification for Bearing Resistance-

i) Load Combination 1

	LC-1	LC-2
	Combination-1	
	LL Lead	Wind Lead
DL, SIDL	1.35	1.35
Surfacing	1.75	1.75
LL & FPLL Load	1.50	1.15
Wind	0.90	1.50
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		**V (t)**	**HT (t)**	**HL (t)**	**MT (t-m)**	**ML (t-m)**	**e_L (m)**	**e_T (m)**	**A_{eff} (m²)**	**% A_{eff}**	**σ_v (t/m²)**
LC-1	+	3455	54	162	1081	1486.	0.430	0.313	136.3	88.5%	25.35
	-	3356	54	162	1081	1486	0.443	0.322	135.8	88.2%	24.71
LC-2	+	3436	76	162	1228	1453	0.423	0.357	135.3	87.9%	25.39
	-	3272	76	162	1228	1453	0.444	0.375	134.4	87.3%	24.35

+ means Wind downward & - means Wind Upward

Max. Base Pressure (P/A_{eff}) = **25.4** t/m² ≤ 72.61 t/sq-m, Safe
Min % Effective Contact Area = **87.27** % ≥ 50 % Safe

(As per Limit State Code)

Characteristic Ultimate bearing capacity = 94.39 t/m²
Design Ultimate bearing capacity (94.39 /1.3) = = 72.61 t/m²

ii) Load Combination 2

	LC-3	LC-4
	Combination-2	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.00	1.00
LL & FPLL Load	1.30	1.00
Wind	0.80	1.30
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	e_L (m)	e_T (m)	A_{eff} (m^2)	% A_{eff}	σ_v (t/m^2)
LC-3	+	2568	47	141	924	1288	0.502	0.360	133.6	86.8%	19.22
	-	2480	47	141	924	1288	0.519	0.372	132.9	86.3%	18.66
LC-4	+	2551	66	141	1043	1259	0.494	0.409	132.5	86.0%	19.25
	-	2408	66	141	1043	1259	0.523	0.433	131.3	85.2%	18.35

+ means Wind downward & - means Wind Upward

Max. Base Pressure (P/A_{eff}) = 19.3 t/m² ≤ 46.32 t/sq-m, Safe
Min % Effective Contact Area = 85.25 % ≥ 50 % Safe

(As per Limit State Code)

Characteristic Ultimate bearing capacity = 60.21 t/m²
Design Ultimate bearing capacity (60.21 /1.3) = 46.32 t/m²

iii) Seismic Combination-

	LC-5A	LC-5B
	Seismic	
	DL Adding	DL Rev.
DL, SIDL	1.35	1.00
Surfacing	1.75	1.00
LL & FPLL Load	0.20	0.20
Wind	-	-
Seismic	1.50	1.50
Buoyancy	1.00	1.00

Factored load	V' (t)	HT' (t)	HL' (t)	MT' (t-m)		ML' (t-m)	
	-	-	Seismic	non Seismic	-	Seismic	non Seismic
LC-5A	3213	87	510	0	682	19	3188
LC-5B	2361	87	510	0	640	19	3188

As the plastic moment is dependent on sectional capacity. It is assumed that section is designed for the exact requirement. Hence the seismic moment and shear at the base is multiplied by over strengthfactor to simplify the process.

Factored load after applying Overstrength Factor of 1.35 in HL & ML -

Factored load	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	e_L (m)	e_T (m)	A_{eff} (m^2)	% A_{eff}	σ_v (t/m^2)
LC-5A	3213	87	688	682	4329	1.348	0.212	119.6	77.63%	26.87
LC-5B	2361	87	688	640	4329	1.834	0.271	108.1	70.17%	21.85

(HL & ML) = (HL' & ML') 1.35*

(V, HT & MT) = (V', HT' & MT')

LC-5A Max. Base Pressure (P/A_{eff}) = **26.9** t/m^2 ≤ 55.41 t/sq-m, Safe

LC-5B Max. Base Pressure (P/A_{eff}) = **≤1.8** t/m^2 ≤ 55.05 t/sq-m, Safe

Min % Effective Contact Area = 70.17% ≥50 % Safe

(As per Limit State Code)

		LC-5A	LC-5B	
Characteristic Ultimate bearing capacity	=	60.95	60.56	t/m^2
Design Ultimate bearing capacity (60.95 /1.1) & (60.56 /1.1)	=	55.41	55.05	t/m^2

3.3 Summary of WSM & LSM-

Method	Description of load case	Loss of contact			Base pressure Check		
		Contact area of base	Allowable contact area of base	Status	Maximum base pressure (t/m^2)	Design ultimate bearing capacity (t/m^2)	Status
Working State Method	Basic	100%	100%	Safe	20.13	38.82	Safe
	Wind				22.43	48.52	Safe
	Seismic				25.47	48.52	Safe

Limit State Method-	SLS	Frequent	100%	90%	Safe	-	-	-
		Rare	-	-	-	21.94	43.13	Safe
	ULS	Load combination-1	87%	50%	Safe	25.39	72.61	Safe
		Load combination-2	85%	50%	Safe	19.25	46.32	Safe
		Seismic Combination	70%	50%	Safe	26.87	55.41	Safe

4. Bearing Capacity Calculations for Soil (As per IS: 6403):-

$$q_d = cN_c s_c d_c i_c + q(N_q - 1)s_q d_q i_q + .5B\gamma N_\gamma s_\gamma d_\gamma i_\gamma W'$$

Length (L) m	=	12.17	
Width (B) m	=	10.73	*(Min of length or width)*
Depth of foundation (D_f) m	=	3.5	
Depth of water Table (Dw) m	=	30	*From ground level.*
Cohesion (c) kN/m²	=	0	
Angle of Resistance (φ)	=	16.23°	
Surcharge (q) (kN/m²)	=	70	
(γ1) bulk kN/m³	=	20	

Net Ultimate Bearing Capacity = *524.05 kN/m²*
(For General Shear Failure because)

Gross Ultimate Bearing Capacity = **60.56** t/m²

	Shape Factor			Depth Factor			Inclination Factor		
Type	Sc	sq	sγ	dc	dq	dγ	ic	iq	iγ
Rectangle	1.176	1.176	0.647	1.09	1.04	1.04	1.00	1.00	1.00

Bearing capacity factor			
Nc	Nq	Nγ	Nφ
11.79	4.43	3.16	1.78

	Lenth (L')	Width (B')	(φ)	GUBC (t/m²)	
Water Table at GL	14.00	11.00	20.00	48.52	WSM:- LC-1 to 3, LSM(SLS):- LC-6 to 10
	12.92	10.10	20.00	46.95	LSM(ULS):- LC-1,2
	12.64	9.88	16.23	29.86	LSM(ULS):- LC-3,4
	10.76	10.49	16.23	29.62	LSM(ULS):- LC-5A
	9.25	10.30	16.23	28.77	LSM(ULS):- LC-5B

	Lenth (L')	Width (B')	(φ)	GUBC (t/m2)	
No Water Table	14.00	11.00	20.00	97.04	WSM:- LC-1 to 3, LSM(SLS):- LC-6 to 10
	13.10	10.24	20.00	94.39	LSM(ULS):- LC-1,2
	12.94	10.12	16.23	60.21	LSM(ULS):- LC-3,4
	12.66	10.79	16.23	60.95	LSM(ULS):- LC-5A
	12.17	10.73	16.23	60.56	LSM(ULS):- LC-5B

Angle of Resistance (φ) = 20.00 Set-1 *From Limit state code*

Angle of Resistance (φ) = 16.23 Set-2 *From Limit state code*

Here L' and B' corresponds to A_{eff} and A gross depending upon case.

5. Prob 3 - Foundation Resting on Rock with Water Table at Ground Level

(A_h & A_t = (Z/2) x (I/R) x (Sa/g) = 0.08)

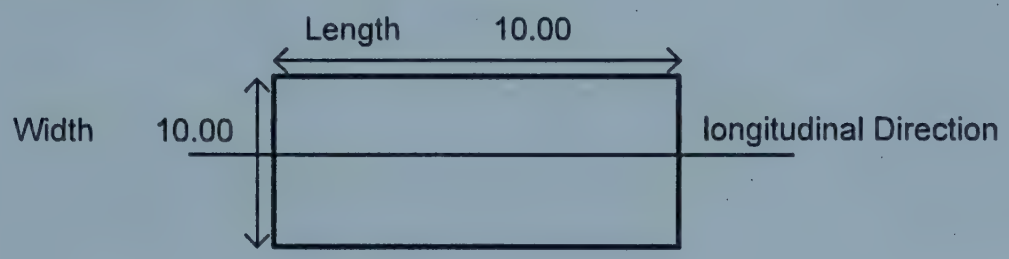


Fig. 4 Plan of Footing

Area of foundation (A_{gross}) = 100.00 m^2

Length of Foundation (in longitudinal Dir.) L = 10.00 m

Width of Foundation (in Transverse Dir.) B = 10.00 m

Section modulus in longitudinal direction, Z_L = 166.67 m^3

Section modulus in transverse direction, Z_T = 166.67 m^3

Summary of Forces of structure without any factor-

For forces refer input data and load calculations (Buoyant & seismic force and footing wt. can be changed depending upon size of footing)

S. No.	Particular	V	H_T	H_L	M_T	M_L
		(t)	(t)	(t)	(tm)	(tm)
1	DEAD LOAD	770.00	-	-	93.74	0.00
2	Surfacing	90.00	-	-	10.96	0.00

3	SIDL	45.00	-	-	5.48	0.00
4	FPLL	0.00	-	-	0.00	0.00
5	LL (Max. V case)	148.38	9.87	-	349.71	96.00
6	Pier cap	140.00	-	-	-	-
7	Pier	42.92	-	-	-	-
8	Footing & cushion	803.17	-	-	-	-
9	Buoyant force	350.00	-	-	-	-
10 (a)	Horizontal force Basic (Live Load case)	-	-	92.38	-	765.22
10 (b)	Horizontal force Basic (No Live load case)	-	-	54.30	-	449.77
10 (c)	Seismic Force	0.00	153.66	304.57	962.27	2066.53
10 (d)	Wind Force with LL Case	54.70	43.16	108.22	448.61	894.75

(While Calculating factored force in seismic longitudinal direction, 30% of seismic vertical and seismic transverse force are also considered.)

5.1 Geo-Technical Design of Foundations using Working Stress Method-

σ_v *(t/m²) = Bearing pressure from trapezoidal pr. Distributions in WSM*

Max. Base Pressure $(P/A+M_L/Z_L+M_T/Z_T)$

Min. Base Pressure $(P/A-M_L/Z_L-M_T/Z_T)$

Partial Safety factors

	As Per IRC:78		
	LC-1	LC-2	LC-3
DL, SIDL	1.00	1.00	1.00
Surfacing	1.00	1.00	1.00
LL & FPLL Load	1.00	1.00	0.20
Wind	0.00	1.00	0.00
Seismic	0.00	0.00	1.00
Buoyancy	1.00	1.00	1.00

i) Basic load combination (LC-1)-

	LC-1
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	1.00
Wind	-
Seismic	-
Buoyancy	1.00

								After taking effect of LOC, if any	
						σ_v (t/m^2)		σ_v'	A_{eff} / A_{Gross}
Factored load	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min	Max	%
	1689	10	92	460	861	24.8	9.0	24.8	100 %

ii) Basic combination with Wind (LC-2)-

	LC-2
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	1.00
Wind	1.00
Seismic	-
Buoyancy	1.00

								After taking effect of LOC, if any	
						σ_v (t/m^2)		σ_v'	A_{eff} / A_{Gross}
Factored load	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min	Max	%
+	1744	53	108	908	991	28.8	6.0	28.84	100 %
-	1635	53	108	908	991	27.7	5.0	27.74	100 %

+ means Wind downward & - means Wind Upward

iii) Seismic Combination (LC-3)-

	LC-2
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	0.20
Wind	-
*Seismic	1.35
Buoyancy	1.00

*Seismic force are multiply by 1.35 as per Clause 219.8 of IRC:6-2017

								After taking effect of LOC, if any	
						σ_v (t/m^2)		σ_v'	A_{eff} / A_{Gross}
Factored load	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min	Max	%
	1571	64	411	570	2809	36.0	-4.6	36.30	94.75 %

Summary of bearing pressure from WSM-

LOC tends to loss of contact if any.

i) Min Pressure		=	-4.57 t/m²	< 0 ,Check for LOC
Effective Contact Area	(For Non Seismic)	=	100 %	> 80% , Safe
	(For Seismic)	=	94.8 %	> 67% , Safe

(Cl 706.3.3.1 of IRC:78-2014)

ii) Max base Pressure

Basic	=	24.8 t/m²	*(≤ 1850 / 7 = 264.29 t/sq-m, Safe)*
Wind	=	28.8 t/m²	*(≤ 1850 * 1.25 / 7 = 330.36 t/sq-m, Safe)*
Seismic	=	36.3 t/m²	*(≤ 1850 * 1.25 / 7 = 330.36 t/sq-m, Safe)*

Gross safe bearing Pressure = 1850 t/m²

5.2 Geo-Technical Design of Foundations using Limit State-

Partial Safety factors

	Table B.4 (IRC: 6-2017)					Table B.3 (IRC: 6-2017)				
	LC-1	LC-2	LC-3	LC-4	LC-5A, 5B	LC-6	LC-7	LC-8	LC-9	LC-10
	Combination-1		Combination-2		Seismic	Frequent		Rare		Quasi
	LL Lead	Wind Lead	LL Lead	Wind Lead	-	LL Lead	Wind Lead	LL Lead	Wind Lead	-
DL, SIDL	1.35	1.35	1.00	1.00	1.35,1	1.00	1.00	1.00	1.00	1.00
Surfacing	1.75	1.75	1.00	1.00	1.75,1	1.20	1.20	1.20	1.20	1.20
LL & FPLL Load	1.50	1.15	1.30	1.00	0.20	0.75	0.20	1.00	0.75	0.00
Wind	0.90	1.50	0.80	1.30	-	0.50	0.60	0.60	1.00	0.00
Seismic	-	-	-	-	1.50	-	-	-	-	-
Buoyancy	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

5.2.1 Serviceability Limit State -

(As per Clause 3.4 Verification of serviceability Limit State)-

σ_v (t/m²) = Bearing pressure from trapezoidal pr. Distributions for SLS check-

Max. Base Pressure $(P/A+M_L/Z_L+M_T/Z_T)$

Min. Base Pressure $(P/A-M_L/Z_L-M_T/Z_T)$

i) Check for Loss of contact-

Frequent Combination	LC-6	LC-7
	Combination-1	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.20	1.20
LL & FPLL Load	0.75	0.20
Wind	0.50	0.60
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	σ_v (t/m^2) Max	σ_v (t/m^2) Min	*After taking effect of LOC, if any* σ_v' Max	*After taking effect of LOC, if any* A_{eff}/A_{Gross} %
LC-6	+	1698	29	81	599	743	25.0	8.9	25.0	100 %
	-	1643	29	81	599	743	24.5	8.4	24.5	100 %
LC-7	+	1622	28	65	451	556	22.3	10.2	22.3	100 %
	-	1556	28	65	451	556	21.6	9.5	21.6	100 %

+ means Wind downward & - means Wind Upward

Min % contact area A_{eff}/A_{Gross} = **100 %** > 80% , Safe

(As per Limit State Code)

ii) Check for Base Pressure-

Rare Combination	LC-8	LC-9
	Combination-2	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.20	1.20
LL & FPLL Load	1.00	0.75
Wind	0.60	1.00
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	After taking effect of LOC, if any: σ_v (t/m²) Max	σ_v (t/m²) Min	σ_v' Max	A_{eff} / A_{Gross} %
LC-8	+	1740	36	108	731	991	27.7	7.1	27.7	100 %
	-	1675	36	108	731	991	27.1	6.4	27.1	100 %
LC-9	+	1725	51	108	823	967	28.0	6.5	28.0	100 %
	-	1616	51	108	823	967	26.9	5.4	26.9	100 %

+ means Wind downward & - means Wind Upward

Max base pressure is = **28.0** t/m² *(≤ 264.29 t/sq.-m, Safe)*
(As per Limit State Code)

Gross safe bearing Pressure = **1850** t/m²

Design Gross safe bearing Pressure (1850 / 7) = **264.3** t/m²

iii) Check for Settlement-

	LC-10
	Quasi
DL, SIDL	1.00
Surfacing	1.20
LL & FPLL Load	0.00
Wind	0.00
Seismic	-
Buoyancy	1.00

Factored load	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	After taking effect of LOC, if any: σ_v (t/m²) Max	σ_v (t/m²) Min	σ_v' Max	A_{eff} / A_{Gross} %
	1559	0	54	112	450	19.0	12.2	19.0	100 %

Settlement to be calculated using = 19.0 t/m²

5.2.2 Ultimate limit state -

σ_v (t/m²) = Bearing pressure from trapezoidal pr. Distributions for ULS check-

Max. Base Pressure $(P/A+M_L/Z_L+M_T/Z_T)$

Min. Base Pressure $(P/A-M_L/Z_L-M_T/Z_T)$

A) Loss of equilibrium-

Overturning, Overall stability and sliding of foundation are carried out under in this clause.

Generally, for pier this check is not required.

B) As per Clause 3.2 Verification for Bearing Resistance-

i) Load Combination 1

	LC-1	LC-2
	Combination-1	
	LL Lead	Wind Lead
DL, SIDL	1.35	1.35
Surfacing	1.75	1.75
LL & FPLL Load	1.50	1.15
Wind	0.90	1.50
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	σ_v (t/m^2) Max	σ_v (t/m^2) Min	*After taking effect of LOC, if any* σ_v' Max	*After taking effect of LOC, if any* A_{eff}/A_{Gross} %
LC-1	+	2511	54	162	1081	1486	40.5	9.7	40.5	100 %
	-	2412	54	162	1081	1486	39.5	8.7	39.5	100 %
LC-2	+	2492	76	162	1228	1453	41.0	8.8	41.0	100 %
	-	2328	76	162	1228	1453	39.4	7.2	39.4	100 %

+ means Wind downward & - means Wind Upward

Max. Base Pressure = **41.0** t/m^2 *(≤ 411.11 t/sq-m, Safe)*

Min % Effective Contact Area = **100** % *> 67% , Safe (As per Limit State Code)*

Gross safe bearing Pressure = **1850** t/m^2

Design Gross safe bearing Pressure (1850 /4.5) = 411.11 t/m^2

ii) Load Combination 2

	LC-3	LC-4
	Combination-2	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.00	1.00
LL & FPLL Load	1.30	1.00
Wind	0.80	1.30
Seismic	-	-
Buoyancy	1.00	1.00

									After taking effect of LOC, if any	
								σ_v (t/m^2)	σ_v'	A_{eff} / A_{Gross}
Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min	Max	%
LC-3	+	1778	47	141	924	1288	31.0	4.5	31.0	100 %
	-	1690	47	141	924	1288	30.2	3.6	30.2	100 %
LC-4	+	1761	66	141	1043	1259	31.4	3.8	31.4	100 %
	-	1618	66	141	1043	1259	30.0	2.4	30.0	100 %

+ means Wind downward & - means Wind Upward

Max. Base Pressure = **31.4** t/m² *(≤ 406.46 t/sq-m, Safe)*

Min % Effective Contact Area = **100** % *> 67% , Safe*
(As per Limit State Code)

Gross safe bearing Pressure = 1321 t/m²

Design Gross safe bearing Pressure (1321 /3.25) 406.46 t/m²

iii) Seismic Combination-

	LC-5A	LC-5B
	Seismic	
	DL Adding	DL Rev.
DL, SIDL	1.35	1.00
Surfacing	1.75	1.00
LL & FPLL Load	0.20	0.20
Wind	-	-
Seismic	1.50	1.50
Buoyancy	1.00	1.00

	V' (t)	HT' (t)	HL' (t)		MT' (t-m)	ML' (t-m)	
Factored load	-	-	*Seismic*	*non Seismic*	-	*Seismic*	*non Seismic*
LC-5A	2269	71	457	0	656	3100	19
LC-5B	1571	71	457	0	613	3100	19

As the plastic moment is dependent on sectional capacity. It is assumed that section is designed for the exact requirement. Hence the seismic moment and shear at the base is multiplied by overstrength factor to simplify the process.

Factored load after applying Overstrength Factor of 1.35 in HL & ML

Factored load	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	After taking effect of LOC, if any			
						σ_v (t/m²)		σ_v'	A_{eff}/A_{Gross}
						Max	Min	Max	%
LC-5A	2269	71	617	656	4211	51.9	-6.5	52.5	93.62 %
LC-5B	1571	71	617	613	4211	44.7	-13.2	50.7	68.97 %

(HL & ML) = (HL' & ML') 1.35*

(V, HT & MT) = (V', HT' & MT')

Max. Base Pressure (P/A_{eff}) = 52.5 t/m² *(≤ 406.46 t/sq-m, Safe)*

Min % Effective Contact Area = 69.0 % *> 67% , Safe*
(As per Limit State Code)

Gross safe bearing Pressure = 1321.00 t/m²

Design Gross safe bearing Pressure (1321 /3.25) = 406.46 t/m²

5.3 Summary of WSM & LSM-

Method		Description of load case	Loss of contact			Base pressure Check		
			Contact area of base	Allowable contact area of base	Status	Maximum base pressure (t/m²)	Design ultimate bearing capacity (t/m²)	Status
Working State Method		Basic	100 %	80%	Safe	24.82	264.29	Safe
		Wind				28.84	330.36	Safe
		Seismic	95 %	67%	Safe	36.30	330.36	Safe
Limit State Method-	SLS	Frequent	100 %	80%	Safe	-	-	-
		Rare	-	-	-	27.99	264.29	Safe
	ULS	Load combination-1	100 %	67%	Safe	41.00	411.11	Safe
		Load combination-2	100 %	67%	Safe	31.42	406.46	Safe
		Seismic Combination	69 %	67%	Safe	52.50	406.46	Safe

6. Prob 4 - Foundation Resting on Rock with no Water Table

(A_h & A_t = (Z/2) x (I/R) x (Sa/g) = 0.08)

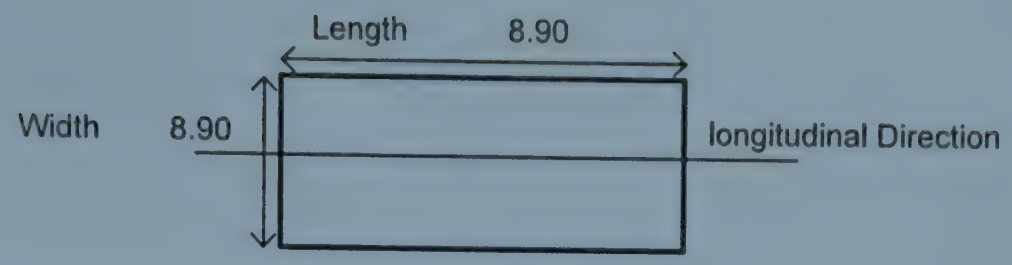


Fig. 5 Plan of Footing

Area of foundation (A_{gross}) = 79.21 m^2
Length of Foundation (in longitudinal Dir.) L = 8.90 m
Width of Foundation (in Transverse Dir.) B = 8.90 m
Section modulus in longitudinal direction, Z_L = 117.49 m^3
Section modulus in transverse direction, Z_T = 117.49 m^3

S. No.	Particular	V	H_T	H_L	M_T	M_L
		(t)	(t)	(t)	(tm)	(tm)
1	DEAD LOAD	770.00	-	-	93.74	0.00
2	Surfacing	90.00	-	-	10.96	0.00
3	SIDL	45.00	-	-	5.48	0.00
4	FPLL	0.00	-	-	0.00	0.00
5	LL (Max. V case)	148.38	9.87	-	349.71	96.00
6	Pier cap	140.00	-	-	-	-
7	Pier	42.92	-	-	-	-
8	Footing & cushion	633.73	-	-	-	-
9	Buoyant force	0.00	-	-	-	-
10 (a)	Horizontal force Basic (Live Load case)	-	-	92.38	-	765.22
10 (b)	Horizontal force Basic (No Live load case)	-	-	54.30	-	449.77
10 (c)	Seismic Force	0.00	140.11	291.02	939.69	2043.96
10 (d)	Wind Force with LL Case	54.70	43.16	108.22	448.61	894.75

(While Calculating factored force in seismic longitudinal direction, 30% of seismic vertical and seismic transverse force are also considered.)

6.1 Geo-Technical Design of Foundations using Working Stress Method-

σ_v *(t/m²) = Bearing pressure from trapezoidal pr. Distributions in WSM*

Max. Base Pressure $(P/A+M_L/Z_L+M_T/Z_T)$

Min. Base Pressure $(P/A-M_L/Z_L-M_T/Z_T)$

Partial Safety factors

	As Per IRC: 78		
	LC-1	LC-2	LC-3
DL, SIDL	1.00	1.00	1.00
Surfacing	1.00	1.00	1.00
LL & FPLL Load	1.00	1.00	0.20
Wind	0.00	1.00	0.00
Seismic	0.00	0.00	1.00
Buoyancy	1.00	1.00	1.00

i) Basic load combination (LC-1)-

	LC-1
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	1.00
Wind	-
Seismic	-
Buoyancy	1.00

Factored load

							After taking effect of LOC, if any		
							σ_v (t/m²)	σ_v'	A_{eff}/A_{Gross}
	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min	Max	%
	1870	10	92	460	861	34.9	12.4	34.9	100 %

ii) Basic combination with Wind (LC-2)-

	LC-2
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	1.00
Wind	1.00
Seismic	-
Buoyancy	1.00

Factored load

							After taking effect of LOC, if any		
							σ_v (t/m²)	σ_v'	A_{eff}/A_{Gross}
	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min	Max	%
+	1925	53	108	908	991	40.5	8.1	40.46	100 %
-	1815	53	108	908	991	39.1	6.8	39.08	100 %

+ means Wind downward & - means Wind Upward

iii) Seismic Combination (LC-3)-

	LC-2
DL, SIDL	1.00
Surfacing	1.00
LL & FPLL Load	0.20
Wind	-
*Seismic	1.35
Buoyancy	1.00

**Seismic force are multiply by 1.35 as per Clause 219.8 of IRC: 6-2017*

							After taking effect of LOC, if any	
						σ_v (t/m²)	σ_v'	A_{eff} / A_{Gross}
Factored load	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min	Max	%
	1751	59	393	561	2779	50.5	-6.3	51.03	94.83 %

Summary of bearing pressure from WSM-

LOC tends to loss of contact if any.

i) Min Pressure	=	-6.31	t/m²	*< 0 ,Check for LOC*
Effective Contact Area (For Non Seismic)	=	100	%	*> 80% , Safe*
(For Seismic)	=	94.8	%	*> 67% , Safe*
				(Cl 706.3.3.1 of IRC:78-2014)

ii) Max base Pressure

Basic	=	34.9	t/m²	(≤ 1850 / 7 = 264.29 t/sq-m, Safe)
Wind	=	40.5	t/m²	(≤ 1850 * 1.25 / 7 = 330.36 t/sq-m, Safe)
Seismic	=	51.0	t/m²	(≤ 1850 * 1.25 / 7 = 330.36 t/sq-m, Safe)

Gross safe bearing Pressure = 1850 t/m²

6.2 Geo-Technical Design of Foundations using Limit State-

Partial Safety factors

	Table B.4 (IRC: 6-2017)					Table B.3 (IRC: 6-2017)				
	LC-1	LC-2	LC-3	LC-4	LC-5A, 5B	LC-6	LC-7	LC-8	LC-9	LC-10
	Combination-1		Combination-2		Seismic	Frequent		Rare		Quasi
	LL Lead	Wind Lead	LL Lead	Wind Lead	-	LL Lead	Wind Lead	LL Lead	Wind Lead	-
DL, SIDL =	1.35	1.35	1.00	1.00	1.35,1	1.00	1.00	1.00	1.00	1.00
Surfacing =	1.75	1.75	1.00	1.00	1.75,1	1.20	1.20	1.20	1.20	1.20
LL & FPLL Load =	1.50	1.15	1.30	1.00	0.20	0.75	0.20	1.00	0.75	0.00

	Table B.4 (IRC: 6-2017)					Table B.3 (IRC: 6-2017)				
	LC-1	LC-2	LC-3	LC-4	LC-5A, 5B	LC-6	LC-7	LC-8	LC-9	LC-10
	Combination-1		Combination-2		Seismic	Frequent		Rare		Quasi
	LL Lead	Wind Lead	LL Lead	Wind Lead	-	LL Lead	Wind Lead	LL Lead	Wind Lead	-
Wind =	0.90	1.50	0.80	1.30	-	0.50	0.60	0.60	1.00	0.00
Seismic =	-	-	-	-	1.50	-	-	-	-	-
Buoyancy =	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

6.2.1 *Serviceability Limit State -*

(As per Clause 3.4 Verification of serviceability Limit State)-

σ_v (t/m^2) = Bearing pressure from trapezoidal pr. Distributions for SLS check-

Max. Base Pressure $(P/A+M_L/Z_L+M_T/Z_T)$

Min. Base Pressure $(P/A-M_L/Z_L-M_T/Z_T)$

i) Check for Loss of contact-

Frequent Combination	LC-6	LC-7
	Combination-1	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.20	1.20
LL & FPLL Load	0.75	0.20
Wind	0.50	0.60
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	σ_v (t/m^2) Max	σ_v (t/m^2) Min	*After taking effect of LOC, if any* σ_v' Max	*After taking effect of LOC, if any* A_{eff}/A_{Gross} %
LC-6	+	1878	29	81	599	743	35.1	12.3	35.1	100 %
	-	1824	29	81	599	743	34.4	11.6	34.4	100 %
LC-7	+	1802	28	65	451	556	31.3	14.2	31.3	100 %
	-	1737	28	65	451	556	30.5	13.3	30.5	100 %

+ means Wind downward & - means Wind Upward

Min % contact area A_{eff}/A_{Gross} = 100 % *> 80%, Safe*

(As per Limit State Code)

ii) Check for Base Pressure-

Rare Combination	LC-8	LC-9
	Combination-2	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.20	1.20
LL & FPLL Load	1.00	0.75
Wind	0.60	1.00
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	σ_v (t/m²) Max	σ_v (t/m²) Min	*After taking effect of LOC, if any* σ_v' Max	*After taking effect of LOC, if any* A_{eff} / A_{Gross} %
LC-8	+	1921	36	108	731	991	38.9	9.6	38.9	100 %
	-	1855	36	108	731	991	38.1	8.8	38.1	100 %
LC-9	+	1906	51	108	823	967	39.3	8.8	39.3	100 %
	-	1796	51	108	823	967	37.9	7.4	37.9	100 %

+ means Wind downward & - means Wind Upward

Max base pressure is = **39.3** t/m² *(≤ 264.29 t/sq-m, Safe)*
(As per Limit State Code)

Gross safe bearing Pressure = **1850** t/m²

Design Gross safe bearing Pressure (1850 / 7) = 264.3 t/m²

iii) Check for Settlement-

Quasi per. Combination	LC-10
	Quasi -
DL, SIDL	1.00
Surfacing	1.20
LL & FPLL Load	0.00
Wind	0.00
Seismic	-
Buoyancy	1.00

								After taking effect of LOC, if any	
						σ_v (t/m²)		σ_v'	A_{eff}/A_{Gross}
Factored load	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min	Max	%
	1740	0	54	112	450	26.7	17.2	26.7	100 %

Settlement to be calculated using = 26.7 t/m²

6.2.2 Ultimate limit state -

σ_v *(t/m²) = Bearing pressure from trapezoidal pr. Distributions for ULS check-*

Max. Base Pressure $(P/A+M_L/Z_L+M_T/Z_T)$

Min. Base Pressure $(P/A-M_L/Z_L-M_T/Z_T)$

A) Loss of equilibrium-

Generally, for pier this check is not required.

B) As per Clause 3.2 Verification for Bearing Resistance-

i) Load Combination 1

	LC-1	LC-2
	Combination-1	
	LL Lead	Wind Lead
DL, SIDL	1.35	1.35
Surfacing	1.75	1.75
LL & FPLL Load	1.50	1.15
Wind	0.90	1.50
Seismic	-	-
Buoyancy	1.00	1.00

									After taking effect of LOC, if any	
							σ_v (t/m²)		σ_v'	A_{eff}/A_{Gross}
Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	Max	Min	Max	%
LC-1	+	2632	54	162	1081	1486	55.1	11.4	55.1	100 %
	-	2534	54	162	1081	1486	53.8	10.1	53.8	100 %
LC-2	+	2613	76	162	1228	1453	55.8	10.2	55.8	100 %
	-	2449	76	162	1228	1453	53.7	8.1	53.7	100 %

+ means Wind downward & - means Wind Upward

Max. Base Pressure = **55.8** t/m² *(≤ 411.11 t/sq-m, Safe)*
Min % Effective Contact Area = **100** % *> 67% , Safe*
(As per Limit State Code)

Gross safe bearing Pressure = **1850** t/m²
Design Gross safe bearing Pressure (1850/4.5) = 411.11 t/m²

ii) Load Combination 2

	LC-3	LC-4
	Combination-2	
	LL Lead	Wind Lead
DL, SIDL	1.00	1.00
Surfacing	1.00	1.00
LL & FPLL Load	1.30	1.00
Wind	0.80	1.30
Seismic	-	-
Buoyancy	1.00	1.00

Factored load		V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	σ_v (t/m²) Max	σ_v (t/m²) Min	*After taking effect of LOC if any* σ_v' Max	*After taking effect of LOC if any* A_{eff}/A_{Gross} %
LC-3	+	1958	47	141	924	1288	43.5	5.9	43.5	100 %
	-	1871	47	141	924	1288	42.4	4.8	42.4	100 %
LC-4	+	1941	66	141	1043	1259	44.1	4.9	44.1	100 %
	-	1799	66	141	1043	1259	42.3	3.1	42.3	100 %

+ means Wind downward & - means Wind Upward

Max. Base Pressure = **44.1** t/m² *(≤ 406.46 t/sq-m, Safe)*
Min % Effective Contact Area = **100** % *> 67%, Safe*
(As per Limit State Code)

Gross safe bearing Pressure = 1321 t/m²
Design Gross safe bearing Pressure (1321/3.25) = 406.46 t/m²

iii) Seismic Combination-

	LC-5A	LC-5B
	Seismic	
	DL Adding	DL Rev.
DL, SIDL	1.35	1.00
Surfacing	1.75	1.00
LL & FPLL Load	0.20	0.20

Wind	-	-
Seismic	1.50	1.50
Buoyancy	1.00	1.00

Factored load	V' (t)	HT' (t)	HL' (t)		MT' (t-m)	ML' (t-m)	
	-	-	*Seismic*	*non Seismic*	-	*Seismic*	*non Seismic*
LC-5A	2390	65	437	0	646	3066	19
LC-5B	1751	65	437	0	603	3066	19

As the plastic moment is dependent on sectional capacity. It is assumed that section is designed for the exact requirement. Hence the seismic moment and shear at the base is multiplied by over strength factor to simplify the process.

Factored load after applying Overstrength Factor of 1.35 in HL & ML -

Factored load	V (t)	HT (t)	HL (t)	MT (t-m)	ML (t-m)	σ_v (t/m²)		*After taking effect of LOC, if any* σ_v'	A_{eff} / A_{Gross}
						Max	Min	Max	%
LC-5A	2390	65	589	646	4165	71.1	-10.8	72.3	90.85 %
LC-5B	1751	65	589	603	4165	62.7	-18.5	71.0	69.17 %

(HL & ML) = (HL' & ML') 1.35*

(V, HT & MT) = (V', HT' & MT')

Max. Base Pressure (P/A_{eff})	=	72.3 t/m²	*(≤ 406.46 t/sq-m, Safe)*
Min % Effective Contact Area	=	69.2 %	*> 67% , Safe (As per Limit State Code)*
Gross safe bearing Pressure	=	1321.00 t/m²	
Design Gross safe bearing Pressure (1321/3.25)	=	406.46 t/m²	

6.3 Summary of WSM & LSM-

Method	Description of load case	Loss of contact			Base pressure Check		
		Contact area of base	Allowable contact area of base	Status	Maximum base pressure (t/m²)	Design ultimate bearing capacity (t/m²)	Status
Working State Method	Basic	100 %	80%	Safe	34.85	264.29	Safe
	Wind				40.46	330.36	Safe
	Seismic	95 %	67%	Safe	51.03	330.36	Safe

Limit State Method-	**SLS**	Frequent	100 %	80%	Safe	-	-	-
		Rare	-	-	-	39.29	264.29	Safe
	ULS	Load combination-1	100 %	67%	Safe	55.80	411.11	Safe
		Load combination-2	100 %	67%	Safe	44.10	406.46	Safe
		Seismic Combination	69 %	67%	Safe	72.29	406.46	Safe

7. Bearing Capacity Calculations for Rock :-

Method using Core Strength (Based on IS : 12070-1987)

The least compressive strength of the core samples of the rocky strata met below the founding level were obtained in laboratory and is as under.

q_c = 7400 t/m^2

Net allowable bearing capacity is estimated by using the following expression

$q_s = q_c * N_j$

q_s = Safe bearing pressure (gross)

q_c = Uniaxial compressive strength of rock core

N_j = Empirical coefficient depending upon the spacing of discontinuities

The equation includes a factor of safety of 3.

The least value of N_j is 0.1 and it pertains to a situation where spacing of discontinuities is 30 to 100 cms. Perusal of the borehole data shows that the least value of q_c of the supporting strata is as under.

q_c = 7400 t/m^2

N_j = 0.25 Assuming Spacing of discontinuities 100 to 300 cm

q_s = 1850 t/m^2

*(7400 * 0.25)*

For Set 2 (Table 2) of Limit State code-

q_c = 7400

q'_c = 5285.71

(7400 / 1.4)

N_j = 0.25

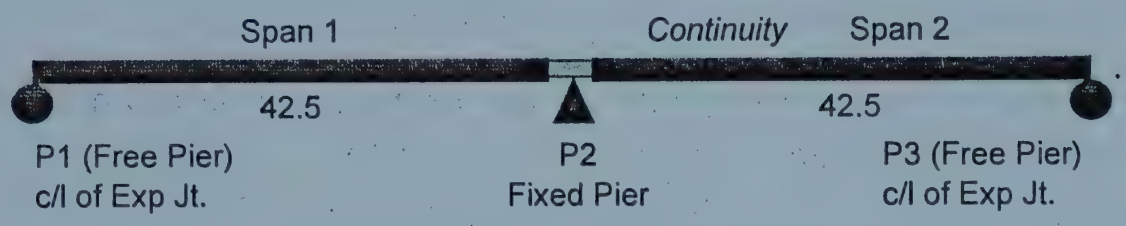
q_s = 1321.43

*(5285.71 * 0.25)*

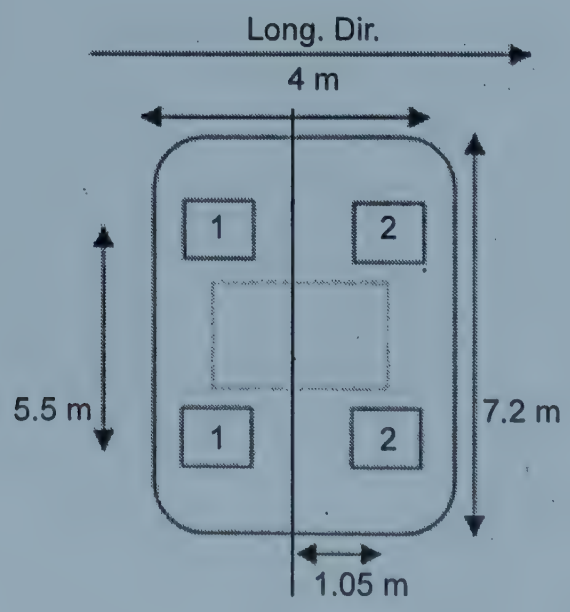
2. EXAMPLE ON PILE FOUNDATION BY LIMIT STATE METHOD AND BY WORKING STRESS METHOD AS PER PRESENT IRC:78

1. INPUT DATA AND LOAD CALCULATIONS

2 Span continuous Module of	42.5 m	+	42.5	m
Pier Height	H	=	9.5	m (Pier Ht. taken as level diff. b/w Pier Cap Top & Pier Base)
Radius of Curvature	R	=	700	m
Design Speed,	V	=	80	km / hr



Depth of Box Girder,	=	2.75	m
Thickness of Wearing Course	=	0.1	m
Bearing Top to Pier Cap Top	=	0.3	m
Depth of Pier Cap	=	2.5	m
Long. Dist. of Bearings from Pier C/l	=	1.05	m
Transverse Dist. b/w Bearings	=	5.5	m
Length of Pier Cap	=	7.2	m (Trans. Direction)
Width of Pier Cap	=	4	m (Long. Direction)
Length of Pier	=	2.7	m (Long. Direction)
Width of Pier	=	2.4	m (Trans. Direction)



Positions of Bearing:

	Left Span	Right Span
Bearing 1	2.75	2.70
Bearing 2	-2.75	-2.80

Material

Material Factor for Concrete γ_m = 1.5 (For Basic & Seismic Case)

Steel γ_s = 1.15

	Pier Cap	Pier	Pile Cap	Pile
Grade of Concrete	**M 50**	**M 50**	**M 35**	**M 35**
f_{ck}	50	50	35	35
f_{cm}	60	60	45	45
E_{cm}	35220	35220	32308	32308
f_{ctm}	3.52	3.52	2.77	2.77
f_{ctk}	2.46	2.46	1.94	1.94
f_{ctd}	1.64	1.64	1.29	1.29
f_{cd}	22.33	22.33	15.63	15.63

Grade of Steel = 500 *MPa*

f_{yd} = 435 *MPa*

E_s = 200000 *MPa*

Cracked MOI = 75%

Density of Concrete = 2.5 t/m^3

Density of Soil Fill = 2 t/m^3

Support Reactions:

Span Length	Dead Load	SIDL Perm	SIDL Surfacing	FPLL + Cycle Track	Live Load w/o Impact - 3 Lane	Live Load w/o Impact - 1 Lane	Vertical Wind	Trans. Wind
42.5	673	53	91	42	124	90	44	24

Impact Factor for Pile = 1

Diameter of Pile = 1.2 m

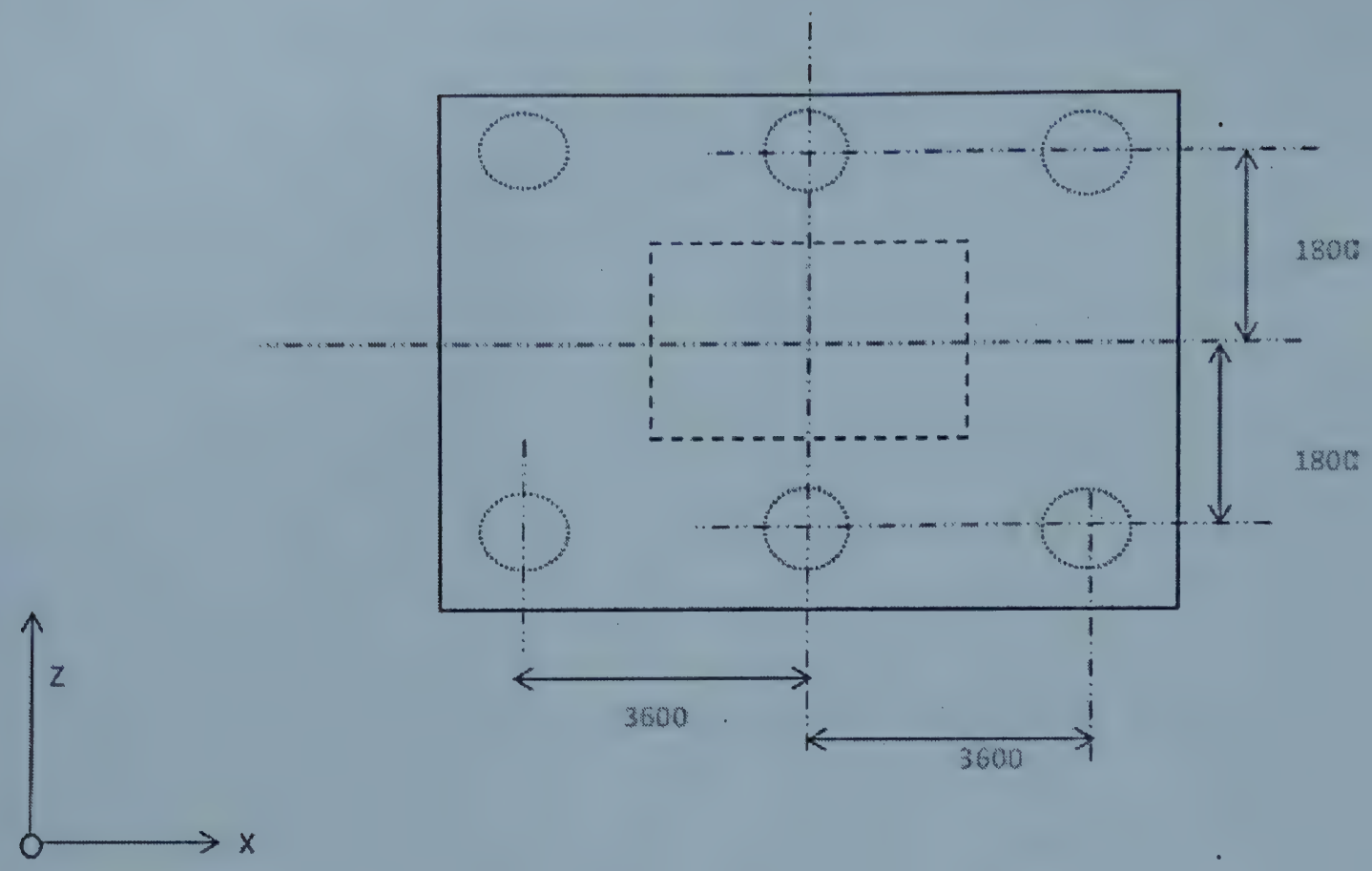
Pile Spacing = 3.6 m

Pile Length = 30 m *(for modelling purpose)*

Depth of Liquefaction zone below GL = 0 m

Top of Pile Cap below GL = 0.5 m

6 Pile Group



No. of Piles	=	6					
Size of pile cap (6 Pile group)	=	8900	X	5300	X	1800	mm
		Long		Trans.		(Depth)	

Idealised STAAD Model For Calculating of Pile Load

Estimation of Seismic Coefficients

Cracked sectional properties (75% of gross properties) have been considered for the analysis as per Cl. 219.5.1 of IRC:6-2014.

Zone Factor,	Z	=	0.24 *(For Seismic Zone IV)*
Importance Factor,	I	=	1.2 *(for Important Bridges)*

For Pile Foundation,

Frequency in Transverse Direction	=	0.721 per sec. ;	Time Period	=	1.387 sec
Frequency in Longitudinal Direction	=	0.768 per sec. ;	Time Period	=	1.301 sec

Sa/g in transverse direction	=	0.981
Sa/g in longitudinal direction	=	1.045

Response Reduction Factor, R

RCC pier with Ductile Detailing (Trans. Direc.)	=	3	
RCC pier with Ductile Detailing (Long. Direc.)	=	3	
Pile Foundations (100% Seismic in Trans. Direc.)	=	2.22	*(As per Table 9 of IRC:6-2014, 35% more than that for Pier, assuming hinge at pier bottom in trans. dir.)*
Pile Foundations (30% Seismic in Long. Direc.)	=	3.00	
Pile Foundations (30% Seismic in Vertical Direc.)	=	3.00	
Pile Foundations (100% Seismic in Long. Direc.)	=	2.22	*(As per Table 9 of IRC:6-2014, 35% more than that for Pier, assuming hinge at pier bottom in long. dir.)*
Pile Foundations (30% Seismic in trans. Direc.)	=	3.00	
Pile Foundations (30% Seismic in Vertical Direc.)	=	3.00	

Plastic hinge is assumed to form at pier bottom in only one direction at a time.

Seismic Acceleration Coefficient, A_h, for Pile Foundation are as follows:

For Piles,

(100% Seismic in trans. Dir.) Seismic Coeff.	A_{hT}	=	0.064 (Plastic hinge in transverse dir.)
(30% Seismic in long. Dir.) Seismic Coeff.	A_{hL}	=	0.050
(30% Seismic in vertical Dir.) Seismic Coeff.	A_{hV}	=	0.031

(100% Seismic in long. Dir.) Seismic Coeff.	A_{hL}	=	0.068 (Plastic hinge in longitudinal dir.)
(30% Seismic in long. Dir.) Seismic Coeff.	A_{hT}	=	0.047
(30% Seismic in vertical Dir.) Seismic Coeff.	A_{hV}	=	0.033

Vertical seismic coefficient are considered as 2/3 of horizontal seismic coefficient.

Total Longitudinal Seismic Force calculated for all the 2 Spans will be applied on the Fixed Pier.

Horizontal Forces on Pier due to Wind Load

As per Cl. 209.3.7 of IRC 6 - 2014, wind speed is restricted to 36 m/s than 39.6 m/s to take live load contribution.

Transverse Pressure,	P_T	=	$P_z \times G \times C_d$	
	P_z	=	$463.7^* (36/27.8)^2$ =	777.59 N/m²
	G	=	2	
	C_d	=	1.3	(For Rect. Pier)
Hence,	P_T	=	2022 N/m²	
		=	0.21 t/m²	

2.0 Load Combinations as Per IRC : 6 - 2017 Table B.4			Pile with Max. Load (KN)	Pile with Min. Load (KN)	
LOAD	1	DEAD LOAD	3138	2656	
LOAD	2	SIDL PERMANENT	355	-23	
LOAD	3	SIDL SURFACING	465	104	
LOAD	4	FPLL + CYCLE TRACK LOAD	526	-263	
LOAD	5	CARRIAGEWAY LL + Centrifugal Force - 3 LANE CASE	878	-101	
LOAD	6	CARRIAGEWAY LL + Centrifugal Force - 1 LANE CASE	919	-356	
LOAD	7	WIND LOAD	713	-438	
LOAD	9	CARRIAGEWAY LL + Centrifugal Force - 3 LANE CASE ON SINGLE SPAN	517	-129	
LOAD	11	Earthquake Transverse - 3 Lane Case	2442	-2442	
LOAD	12	Earthquake Transverse - 1 Lane Case	2422	-2422	
LOAD	13	Earthquake Long.	4035	-4035	For R=1
LOAD	14	Earthquake Vertical - 3 Lane Case	401	277	
LOAD	15	Earthquake Vertical - 1 Lane Case	404	270	
Limit State As Per Table B4					
Load Combination-1					
Live Load Leading	L1*1.35+L2*1.35+L3 *1.75+(L4+L5)*1.5+L7*0.9		8277 kN		
Wind Load Leading	L1*1.35+L2*1.35+L3 *1.75+(L4+L5)*1.15+L7*1.5		8213 kN		
Load Combination-2					
Live Load Leading	L1*1.0+L2*1.0+L3 *1.0+(L4+L5)*1.3+L7*0.8		6353 kN		
Wind Load Leading	L1*1.0+L2*1.0+L3 *1.0+(L4+L5)*1+L7*1.3		6289 kN		
Seismic Combination					
Seismic Trans Servic	L1*1.35+L2*1.35+L3*1.75+L7*0.0+(L11+0.3*L13+0.3*L14)*1.5		7843 kN		
Seismic Long Servic	L1*1.35+L2*1.35+L3*1.75+L7*0.0+(L13+0.3*L11+0.3*L14)*1.5		8680 kN		
Working Stress Method as per Table 1					
Normal Case	L1+L2+L3 +(L4+L5)+L7		6075 kN		
Seismic Case Load Combination VI					
Seismic Trans Servic	L1+L2+L3 +(L4+L5)*0.2+L7+ L11+0.3*L13+0.3*L14		6494 kN		
Seismic Long Servic	L1+L2+L3 +(L4+L5)*0.2+L7+ L13+0.3*L11+0.3*L14		7052 kN		

3.0 Presentation of Results For Pile Foundation Solved Example:

Load Combination Limit State As Per Table B4	Applied Action (KN)	*Pile Resistance (KN)	
		φ= 25 , Pile Length = 29m	φ = 30 , Pile Length = 29m
Load Combination-1			
Live Load Leading	8277	12286	16985
Wind Load Leading	8213	12286	16985
Load Combination-2			
Live Load Leading	6353	7227	9991
Wind Load Leading	6289	7227	9991
Seismic Combination			
Seismic Trans Service	7843	8775	12132
Seismic Long Service	8680	8775	12132
Working Stress Method as per Table 1			
Normal Case	6075	5897	8153
Seismic Case Load Combination VI			
Seismic Trans Service	6494	7371	10191
Seismic Long Service	7052	7371	10191

For pile capacities refer subsequent calculations

4.0 Calculations of Pile Capacity using Ground Parameters with angle of friction φ=25⁰:

(Taking soil stratification into account)

Existing G.L.	=	***0***	*m*	*R.L. (Assumed as 0.00m for simplicity)*
Pile Length	=	***28.70***	***m***	
Working Load at Cut-off Level	=	***0***	***kN***	
Working Pile capacity (Theoretically calculated)	=	***5897***	***kN***	
Base resistance	=	***1115***	***kN***	**19 % of total capacity**
Shaft resistance	=	***4782***	***kN***	**81 % of total capacity**
Founding level of pile	=	*-31.000*	*m*	*R.L.*

INPUT DATA :

Table 2

Partial Factor for Soil Parameters (γ_m) for computing the geo technical actions and bearing capacity

Soil Parameter	Symbol	Set 1	Set 2
		To be used in Combination (1) and in Accidental Combination	To be used in Combination (2) and in seismic combination
Angle of Shearing Resistance	φ	1.0	1.25
Effective Cohesion	γ_c	1.0	1.25
Undrained Shear Strength	γ_{cu}	1.0	1.4
unconfined compressive strength	γ_{q3}	1.0	1.4
Density	γ_t	1.0	1.0

Design angle of shearing resistance=$\varphi^1=\tan^{-1}(\tan\varphi/\gamma_m)$

Table 4 Set Values of Partial Factor (γ_m) to be used for Material for Computation of Geo Technical action

Combination	Set
For computation of geo technical action, negative skin friction and estimation of effects due to geotechnical lateral actions including those specified in Clause 4.2 for combination A1	1
For computation of Geo technical action in Combination A2 and in seismic combination excluding the negative skin friction effects and effects due to geo technical lateral actions including those specified in Clause 4.2	1
In Combination A2 and in seismic combination for computation of negative skin friction effects and effects due to geo technical lateral action, including those specified in Clause 4.2	2
For accidental combination for estimation of geotechnical action inclusive of negative skin friction effects	1
For accidental combination for estimation of effects of lateral actions including those specified in Clause 4.2	2

Table 5

Partial Factor for ground resistance for shaft in compression

Component	Symbol	For bored and continuous auger Pile		Driven Piles		GRS for Seismic Combination on	GRA for accidental combination		Modal Factor for bored and auger pile and driven piles
		GR1	GR2	GR1	GR2		For Vehicle collision Log Impact and barge collision	Ship Collision	M
For Base resistance	γ_b	1.0	1.7	1.0	1.5	1.40	1.25	1.40	1.2
For shaft resistance	γ_s	1.0	1.4	1.0	1.3	1.15	1.25	1.15	1.2
Total combined resistance	γ_t	1.0	1.7	1.0	1.5	1.40	1.25	1.40	1.2

Ground Level From Bore Hole	=	**0.000**	*m*
Diameter of pile, D	=	**1.2**	*m*
Max. Depth for Overburden Pr. = 20 x Pile dia.	=	**24**	*m*
Pile cap Top Level	=	-0.500	*m*
Pile cap Bottom LVL	=	-2.300	*m*
Depth of Water Table	=	**0.000**	*m*
Submerged Pile Length	=	**28.70**	*m*

INPUT SOIL PARAMETERS :

Soil Layer No. As per approved GIR parameter table	Level at Top of soil Layer from assumed G.L. (0.00m)	Level at Bottom of soil Layer from assumed G.L. (0.00m)	Wet/ Bulk Density	Cohesion	Angle of internal friction	Effective Density of soil	Earth Pressure Coefficient	Reduction factor (as per fig. 2 of IS 2911 (Part 1/ Sec 2) : 2010
				C	$\Phi = \delta$	γ_{eff}	K	α
	(m)	(m)	(kN/m^3)	(kN/m^2)		(kN/m^3)		
0	0.000	1.000	18.0	0	0.0	10.000	1.500	0
0	1.000	2.300	20.0	0.00	0.0	10.000	1.500	0
1	2.300	8.000	20.0	0.00	25.0	10.000	1.500	0.4
2	8.000	9.000	20.0	0.00	25.0	10.000	1.500	0.35
3	9.000	11.000	20.0	0.00	25.0	10.000	1.500	0.4
4	11.000	14.000	20.0	0.00	25.0	10.000	1.500	0.55
4	14.000	14.500	20.0	0.00	25.0	10.000	1.500	0.35
5	14.500	18.000	20.0	0.00	25.0	10.000	1.500	0.35
6	18.000	17.000	20.0	0.00	25.0	10.000	1.500	0.35
7	17.000	20.000	20.0	0.00	25.0	10.000	1.500	0.27
7	20.000	24.000	20.0	0.00	25.0	10.000	1.500	0.27
8	24.000	31.000	20.0	0.00	25.0	10.000	1.500	0

(1) SHAFT RESISTANCE

Soil Layer No. As per approved GIR parameter table	Depth for Over burden Pressure from G.L.	PD at Top of Layer	PD at bottom of Layer	Thickness of Layer	PDi at centre of layer	tan δ	Surface Area Asi	K*PDi* tanδ* Asi	α*C* Asi
	(m)	(kN/m^2)	(kN/m^2)	(m)	(kN/m^2)		m^2	kN	kN
0	1.000	0.000	10.000	1	5.0	0.000	3.770	0	0.00
0	2.300	10.000	23.000	1.30	16.5	0.000	4.901	0.00	0.00
1	8.000	23.000	80.000	5.70	51.5	0.466	21.488	774	0.00
2	9.000	80.000	90.000	1.00	85.0	0.466	3.770	224	0.00
3	11.000	90.000	110.000	2.00	100.0	0.466	7.540	527	0.00
4	14.000	110.000	140.000	3.00	125.0	0.466	11.310	989	0.00
4	14.500	140.000	145.000	0.50	142.5	0.466	1.885	188	0.00
5	18.000	145.000	180.000	3.50	162.5	0.466	13.195	1500	0.00
6	17.000	180.000	170.000	-1.00	175.0	0.466	-3.770	-461	0.00
7	20.000	170.000	200.000	3.00	185.0	0.466	11.310	1463	0.00
7	24.000	200.000	240.000	4.00	220.0	0.466	15.080	2320	0.00
8	24.000	240.000	240.000	7.00	240.0	0.466	26.389	4430	0.00
							Total	**11955**	**0**

Total Ultimate Shaft resistance = **11955 *kN***

(2) BASE RESISTANCE :

Angle of friction at Toe	Φ	=	25°		
Bearing Capacity Factors :					
Refer Anexure B-2, IS : 2911 part 1/sec 2 for	**Nc**	=	20.72	Value From Graph	
Refer fig 1, IS : 2911 part 1/ sec 2 for	**Nq**	=	10.00	Φ	Nq
Refer Table 1, IS : 6403 for Ng	**Nγ**	=	10.88	30	20
Cross-sec.area of pile toe, Ap		=	1.13 m^2	31	24.5
Eff. unit wt.of soil at pile toe, g		=	10.00 kN/m^3	32	29
Eff. overburden at pile toe, PD		=	240.00 kN/m^2	33	33
Average cohesion at pile toe, Cp		=	0.00 kN/m^2	34	41
				35	47

Qu1 = Ap*(0.5*D*g*Ng + PD*Nq) = 73.83 *kN* + 2714.3 *kN* = **2788 *kN***

Qu2 = Ap*Nc*Cp = **0 kN**

Total resistance due to end bearing = **2788 kN**

A. Pile Capacity Based On Working Stress Method

Factor of Safety = 2.5

Capacity Load at Pile Toe = 5897 **kN**

B. Pile Capacity Based On Limit State:

CAPACITY OF PILE:

		Q_{ufc} Skin Friction		Q_{ubc} End Bearing		Q_{uc} Total	
Ultimate Bearing Capacity of Pile	=	11955	+	2788	=	**14743**	***kN***
		81%		**19%**			
Q_{Rd}	=	Q_{ubc}/γ_b	+	Q_{ufc}/γ_s	or	Q_{uc}/γ_t	

Where Q_{uc} is the characteristic resistance

Q_{ubc} characteristic base resistance

Q_{ufc} characteristic shaft resistance

Q_{Rd} is the reduced capacity of pile based on partial factor for ground resistance.

For Load Combination 1 & Accidental Load Combination

Q_{Rd}	=	11955/1	+	2789/1	or	14743/1	
		11955	+	2788	or	14743	**kN**
Q_{ud}	=	Q_{Rd}/M =	$Q_{Rd}/1.2$	(Capacity of Pile)	=	12286	**kN**

For Load Combination 2

Q_{Rd}	=	11955/1.4	+	2789/1.7	or	14743/1.7		
Q_{Rd}	=	8539	+	1640	or	8672		**kN**
Q_{ud}	=	Q_{Rd}/M =	Q_{Rd}/1.2	(Capacity of Pile)		=	7227	**kN**

For Seismic Load Combination

Q_{Rd}	=	11955/1.15	+	2789/1.4	or	14743/1.4		
Q_{Rd}	=	10395	+	1992	or	10531		**kN**
Q_{ud}	=	Q_{Rd}/M =	Q_{Rd}/1.2	(Capacity of Pile)		=	8775	**kN**

5.0 Calculations of Pile Capacity using Ground Parameters with angle of friction φ=30⁰:

(Taking soil stratification into account)

Existing G.L.	=	0	*m R.L.* (Assumed as 0.00m for simplicity)	
Pile Length	=	**28.70**	***m***	
Working Load at Cut-off Level	=	**0**	**kN**	
Working Pile capacity (Theoretically calculated)	=	**8153**	**kN**	
Base resistance	=	**2232**	**kN**	**27 % of total capacity**
Shaft resistance	=	**5921**	**kN**	**73 % of total capacity**
Founding level of pile	=	-31.000	*m R.L.*	

INPUT DATA :

Table 2

Partial Factor for Soil Parameters (γ_m) for computing the geo technical actions and bearing capacity

Soil Parameter	Symbol	Set 1	Set 2
		To be used in Combination (1) and in Accidental Combination	To be used in Combination (2) and in seismic combination
Angle of Shearing Resistance	φ	1.0	1.25
Effective Cohesion	γ_c	1.0	1.25
Undrained Shear Strength	γ_{cu}	1.0	1.4
unconfined compressive strength	γ_{q3}	1.0	1.4
Density	γ_t	1.0	1.0

Design angle of shearing resistance=$\varphi^1=\tan^{-1}(\tan\varphi/\gamma_m)$

Table 4 Set Values of Partial Factor (γ_m) to be used for Material for Computation of Geo Technical action

Combination	Set
For computation of geo technical action, negative skin friction and estimation of effects due to geotechnical lateral actions including those specified in Clause 4.2 for combination A1	1
For computation of Geo technical action in Combination A2 and in seismic combination excluding the negative skin friction effects and effects due to geo technical lateral actions including those specified in Clause 4.2	1
In Combination A2 and in seismic combination for computation of negative skin friction effects and effects due to geo technical lateral action, including those specified in Clause 4.2	2
For accidental combination for estimation of geotechnical action inclusive of negative skin friction effects	1
For accidental combination for estimation of effects of lateral actions including those specified in Clause 4.2	2

Table 5

Partial Factor for ground resistance for shaft in compression

Component	Symbol	For bored and continuous auger Pile		Driven Piles		GRS for Seismic Combination on	GRA for accidental combination		Modal Factor for bored and auger pile and driven piles
		GR1	GR2	GR1	GR2		For Vehicle collision Log Impact and barge collision	Ship Collision	M
For Base resistance	γ_b	1.0	1.7	1.0	1.5	1.40	1.25	1.40	1.2
For shaft resistance	γ_s	1.0	1.4	1.0	1.3	1.15	1.25	1.15	1.2
Total combined resistance	γ_t	1.0	1.7	1.0	1.5	1.40	1.25	1.40	1.2

Ground Level From Bore Hole	=	**0.000** ***m***
Diameter of pile, D	=	**1.2** ***m***
Max. Depth for Overburden Pr. = 20 x Pile dia.	=	**24** ***m***
Pile cap Top Level	=	-0.500 *m*
Pile cap Bottom LVL	=	-2.300 *m*
Depth of Water Table	=	**0.000** ***m***
Submerged Pile Length	=	**28.70** ***m***

INPUT SOIL PARAMETERS :

Soil Layer No. As per approved GIR parameter table	Level at Top of soil Layer from assumed G.L. (0.00m)	Level at Bottom of soil Layer from assumed G.L. (0.00m)	Wet/ Bulk Density	Cohesion	Angle of internal friction	Effective Density of soil	Earth Pressure Coefficient	Reduction factor (as per fig. 2 of IS 2911 (Part 1/ Sec 2) : 2010
				C	$\Phi = \delta$	γ_{eff}	K	α
	(m)	(m)	(kN/m^3)	(kN/m^2)		(kN/m^3)		
0	0.000	1.000	18.0	0	0.0	10.000	1.500	0
0	1.000	2.300	20.0	0.00	0.0	10.000	1.500	0
1	2.300	8.000	20.0	0.00	30.0	10.000	1.500	0.4
2	8.000	9.000	20.0	0.00	30.0	10.000	1.500	0.35
3	9.000	11.000	20.0	0.00	30.0	10.000	1.500	0.4
4	11.000	14.000	20.0	0.00	30.0	10.000	1.500	0.55
4	14.000	14.500	20.0	0.00	30.0	10.000	1.500	0.35
5	14.500	18.000	20.0	0.00	30.0	10.000	1.500	0.35
6	18.000	17.000	20.0	0.00	30.0	10.000	1.500	0.35
7	17.000	20.000	20.0	0.00	30.0	10.000	1.500	0.27
7	20.000	24.000	20.0	0.00	30.0	10.000	1.500	0.27
8	24.000	31.000	20.0	0.00	30.0	10.000	1.500	0

(1) SHAFT RESISTANCE

Soil Layer No. As per approved GIR parameter table	Depth for Over burden Pressure from G.L.	PD at Top of Layer	PD at bottom of Layer	Thickness of Layer	PDi at centre of layer	tan δ	Surface Area Asi	K*PDi* tanδ* Asi	α*C* Asi
	(m)	(kN/m²)	(kN/m²)	(m)	(kN/m²)		m²	kN	kN
0	1.000	0.000	10.000	1	5.0	0.000	3.770	0	0.00
0	2.300	10.000	23.000	1.30	16.5	0.000	4.901	0.00	0.00
1	8.000	23.000	80.000	5.70	51.5	0.577	21.488	958	0.00
2	9.000	80.000	90.000	1.00	85.0	0.577	3.770	278	0.00
3	11.000	90.000	110.000	2.00	100.0	0.577	7.540	653	0.00
4	14.000	110.000	140.000	3.00	125.0	0.577	11.310	1224	0.00
4	14.500	140.000	145.000	0.50	142.5	0.577	1.885	233	0.00
5	18.000	145.000	180.000	3.50	162.5	0.577	13.195	1857	0.00
6	17.000	180.000	170.000	-1.00	175.0	0.577	-3.770	-571	0.00
7	20.000	170.000	200.000	3.00	185.0	0.577	11.310	1812	0.00
7	24.000	200.000	240.000	4.00	220.0	0.577	15.080	2873	0.00
8	24.000	240.000	240.000	7.00	240.0	0.577	26.389	5485	0.00
							Total	**14801**	0

Total Ultimate Shaft resistance = **14801 *kN***

(2) BASE RESISTANCE :

Angle of friction at Toe Φ = 30°

Bearing Capacity Factors :

Refer Anexure B-2, IS : 2911 part 1/sec 2 for Nc = 30.14

Refer fig 1, IS : 2911 part 1/ sec 2 for Nq = 20.00

Refer Table 1, IS : 6403 for Ng Nγ = 22.40

Cross-sec.area of pile toe, Ap = 1.13 m²

Eff. unit wt.of soil at pile toe, g = 10.00 kN/m³

Eff. overburden at pile toe, PD = 240.00 kN/m²

Average cohesion at pile toe, Cp = 0.00 kN/m²

Value From Graph	
Φ	Nq
30	20
31	24.5
32	29
33	33
34	41
35	47

Qu1 = Ap*(0.5*D*g*Ng + PD*Nq) = 152.00 *kN* + 5428.7 *kN* = **5581 *kN***

Qu2 = Ap*Nc*Cp = **0 kN**

Total resistance due to end bearing = **5581 kN**

A. Pile Capacity Based On Working Stress Method

Factor of Safety	=	2.5
Capacity Load at Pile Toe	=	8153 *kN*

B. Pile Capacity Based On Limit State:

CAPACITY OF PILE:

		Q_{ufc} Skin Friction		Q_{ubc} End Bearing		Q_{uc} Total	
Ultimate Bearing Capacity of Pile	=	14801	+	5581	=	**20382**	***kN***
		73%		**27%**			
Q_{Rd}	=	Q_{ubc}/γ_b	+	Q_{ufc}/γ_s	or	Q_{uc}/γ_t	

Where Q_{uc} is Q_{ubc} the characteristic Resistance

characteristic base resistance

Q_{ufc} Characteristic shaft resistance

Q_{Rd} is the reduced capacity of pile based on partial factor for ground resistance.

For Load Combination 1 & Accidental Load Combination

Q_{Rd}	=	14802/1	+	5581/1	or	20382/1	
		14801	+	5581	or	20382	**kN**
Q_{ud}	=	Q_{Rd}/M =	$Q_{Rd}/1.2$	(Capacity of Pile)	=	16985	**kN**

For Load Combination 2

Q_{Rd}	=	14802/1.4	+	5581/1.7	or	20382/1.7	
Q_{Rd}	=	10572	+	3283	or	11989	**kN**
Q_{ud}	=	Q_{Rd}/M =	$Q_{Rd}/1.2$	(Capacity of Pile)	=	9991	**kN**

For Seismic Load Combination

Q_{Rd}	=	14802/1.15	+	5581/1.4	or	20382/1.4	
Q_{Rd}	=	11841	+	3986	or	14559	**kN**
Q_{ud}	=	Q_{Rd}/M =	$Q_{Rd}/1.2$	(Capacity of Pile)	=	12132	**kN**

6.0 Conclusion & Discussion

Capacity of soil =	12286x1.2 =	**14743**	**kN**
With FOS 2.5 allowable load =	14743/2.5 =	5897	**kN** *(Figure matches with working stress-Normal)*
For seismic case :	14743/2.0 =	7371 **kN**	*(Figure matches with seismic capacity)*
For combination-1 :	14743/1.2 =	12286 **kN**	(Matches)
For combination-2:	14743/(1.7x1.2) =	7227 **kN**	(matches) (FOS = 2.04)

Alternative check. With FOS 2.5 capacity 5897 kN With 2.04 it should be 2.5/2.04x5897=7227 kN (matches)

During seismic FOS is 14743/8775 = 1.68 (2.04x0.8=1.63 Figure matches)

The design is governed by Seismic case 8775 kN(capacity) versus 8680 kN

Then under this situation for combination 2 the FOS will increase and will be 2.04x 7227/6353=2.32. Very close to 2.5. The FOS will become 2.32

Conclusion

FOS for combination-2 is 2.32

FOS for Seismic case is 1.68

3. EXAMPLE OF PIER WELL FOUNDATION BY WORKING STRESS METHOD AS PER IRC:78 AND LIMIT STATE METHOD

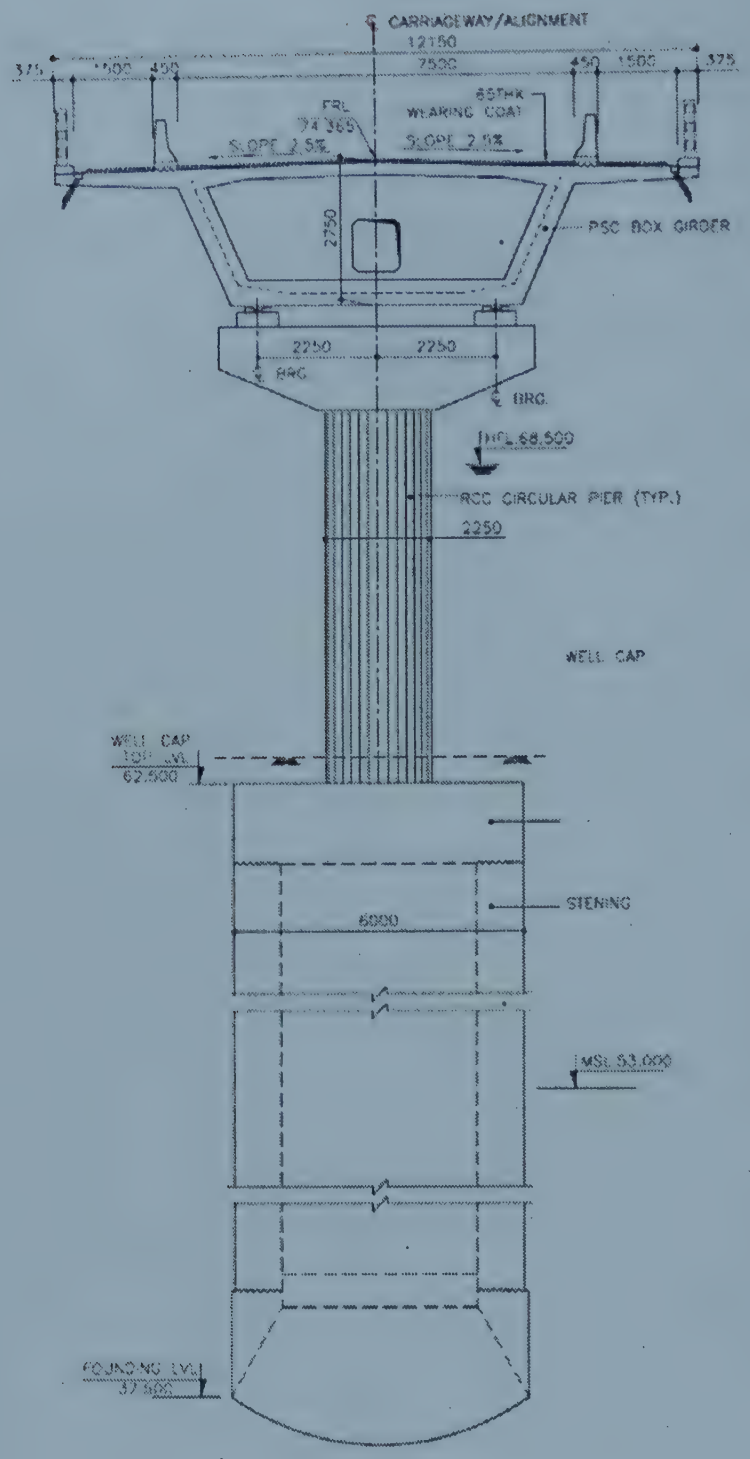
1. INPUT DATA AND LOAD CALCULATION

1.1 SAILENT LEVELS

Type of superstructure arrangement		ON BEARING
Span fix on pier / left span	=	43.000 m
Span free on pier / right span	=	43.000 m
Radius of curvature	=	650.000 m
FRL	=	74.365 m
Total width of the bridge	=	12.150 m
Total carriageway & footpath	=	11.250 m
Bearing level	=	71.565 m
CG of bearing pad	=	71.965 m
Pier cap top level	=	71.065 m
Depth of pier cap	=	2.000 m
Bottom of pier cap	=	69.065 m
Top of pier	=	69.065 m
Top of well cap	=	62.500 m
Founding level (Limit state)	=	34.000 m
Founding level (Working stress)	=	34.000 m
High flood level	=	68.500 m
Low water level	=	61.000 m
Max. Scour level	=	53.000 m
Max. Scour level (seismic)	=	54.550 m
Bed level	*SOIL* =	66.500 m

1.2 Bearing Capacity Table: (Limit State Design)

	LC-1	LC-2	LC-3	LC-4	LC-5A, 5B	LC-6	LC-7
	Action Combination-1 Set-1		Action Combination-2 Set-2		Seismic Set-2	Accidental Set-2	Quasi
Net Bearing Capacity t/m²	971		469.61		469.61		469.61
Partial Factor for material for base resistance	1.35	1.35	1.10	1.10	1.00	1.00	1.00
Design bearing capacity	719	719	427	427	470	470	470



Net Bearing Capacity (For Working Stress) = 90 t/m²
Gross Bearing Capacity (Obtained from Geotech Report) = 109 t/m²

1.3 LOAD CALCULATION:

(a) ***Dead Load***

Total dead load of 43 m span = 800.00 t

(b) ***Superimposed Dead Load***

Total superimposed deal load for 43 m span = 68.80 t
= 70.00 t (Say)

(C) ***RCC Wearing coat*** = 0.20 t/m²

Total wt. of wearing coat/surfacing for 43 m span = 96.75 t
= 100.00 t (Say)

Total weight of pier cap = 100.00 t
Total weight of pier = 65.3 t

(d) Loads due to LL:

Case	Loads from (t)		Dst. Btw cl exp. Joint & cl brg (m)		Longitudinal moment (tm)	Transverse moment (tm)	Remarks
	Free/ Right span	Fixed/ Left span	Free/ Right span	Fixed/ Left span			
Max. V case	45.71	119.29	1.05	1.05	77.25	175.24	Due to three lanes of class A
Max. ML case	0.00	119.23			125.19	241.48	Due to single lane of class 70RW & class A
Max. MT case	32.84	107.96			78.88	321.05	Due to single lane of class 70RW & class A

(e) Centrifugal Force

Centrifugal force = $W.V^2/127R$

where,

W is the wt. on pier

V is the speed of vehicle

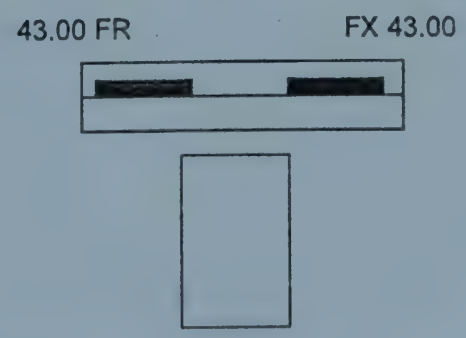
R is the radius of curvature

Speed of vehicle = 80 km/hr

Case	W (t)	Centrifugal force (t)	M_T (tm)
			Pier Base
Max. V case	165.00	12.79	167.13
Max. ML case	119.23	9.24	120.77
Max. MT case	140.81	10.92	142.62

(f) Horizontal force on a pier in normal condition:

Arrangement to transfer horizontal force *POT-PTFE*



Applied horizontal force: As per Clause 211.2 (b) of IRC:6-2014

For span of 43 m : Either two lanes of Class A or Class 70RW can accommodate on this span

For span of 43 m : Either three lanes of Class A can accommodate on this span

Therefore total horizantal force for individual spans:

			On 43 m	On 43 m
With single lane of Class A & Class 70RW,	F_h	=	22.77	22.77 t
With three lanes of Class A,	F_h	=	13.85	13.85 t
Governing	F_h	=	22.77	22.77 t

(i) *Transfering Horizontal Force Through POT-PTFE Bearings*

As per Clause 211.5.1.1 of IRC:6-2014,

Force on fixed bearing = $F_h - \mu2*(R_g + R_q)$ OR $F_h/2 + \mu1*(R_g + R_q)$

Force on free bearing = $\mu2*(R_g + R_q)$

Where,

F_h	=	Applied horizontal force		
R_g	=	Reaction due to dead load		
R_q	=	Reaction due to live load		
μ	=	Coefficient of friction at movable bearings	$\mu1$	0.05
			$\mu2$	0.03

Horizontal force on fixed bearing	=	39.68 t
R_g	=	485.00 t
R_q	=	80.99 t
Total force on a pier	=	39.68 t

(Force on free bearing is ignored as of opposite direction)

(g) Seismic Forces

	=	IV

(g) 1. Longitudinal Seismic Coefficient

Zone factor for seismic zone IV, Z	=	0.24
Time period	=	0.60 sec
For medium soil (10>N>30), Sa/g = 1.36/T	=	2.3
Importance Factor, I	=	1.5
Response Reduction Factor, R	=	3
$A_h = (Z/2) \times (I/R) \times (Sa/g)$	=	0.136

(g) 2. Transverse Seismic Coefficient

Zone factor for seismic zone IV, Z	=	0.24
Time period	=	0.7 sec
For medium soil (10>N>30), Sa/g = 1.36/T	=	1.8
Importance Factor, I	=	1.5
Response Reduction Factor, R	=	3

A_h = (Z/2) x (I/R) x (Sa/g) = 0.110

Vertical Seismic Coefficient = 2/3 Horizontal Seismic Coefficient = *0.091*

(g) 3. Horizontal Forces in Seismic Condition

**For seismic load calculations, consider only 20% LL* 0.20

(i) Seismic Transverse Case With POT-PTFE

Long. seismic on superstructure (30% seismic long)		39.58	t
	μ1	0.05	
	μ2	0.03	
Frictional force on free bearing	μ2(Rg)	14.55	t
Frictional force on fixed bearing	F_h-μ2(Rg)	25.03	t
	or F_h/2+μ1(Rg+Rq)	44.85	t
	or F_h	39.58	t

**Force on free bearing is acting opposite to the force on fixed bearing, hence ignored to be on conservative side.*

Total force on a pier **44.85 t**

Thus, max force on a pier **44.85 t**

(ii) Seismic Longitudinal Case With POT-PTFE

Seismic force on superstructure (Fixed Span)		131.92	t
	μ1	0.05	
	μ2	0.03	
Frictional force on free bearing	μ2(Rg)	14.55	t
Frictional force on fixed bearing	F_h-μ2(Rg)	117.37	t
	or F_h/2+μ1(Rg+Rq)	91.02	t
	or F_h	131.92	t

** Force on free bearing is acting opposite to the force on fixed bearing, hence ignored to be on conservative side.*

Total force on a pier **131.92 t**

Thus, max force on a pier **131.92 t**

(g) 4. In Longitudinal Direction

Longitudinal Sesmic Force Due to	Force (t)	Lever arm (m)	Moment (tm)
Superstructure	131.92	9.07	1195.85
Pier cap	13.60	7.68	104.46
Pier	8.88	3.28	29.13
Total longitudinal Seismic force	**154.40**		**1329.44**

(g) 5. In Transverse Direction

Transverse Sesmic Force Due to	Force (t)	Lever arm (m)	Moment (tm)
Live Load (Horizontal) at 1.2 above FRL	3.64	13.07	47.54
Dead Load (Free span)	44.11	10.82	477.03
Dead Load (Fixed span)	44.11	10.82	477.03
Surfacing	11.03	11.87	130.84
SIDL	7.72	12.37	95.44
Pier cap	11.03	7.68	84.70
Pier	7.20	3.28	23.62
Total transverse Seismic force	**128.82**		**1336.20**

(g) 6. Summary of Seismic Forces & its Moment

	Force (t)	Moment (tm)
Transverse Direction	128.82	1336.20
Longitudinal Direction	154.40	1329.44
Vertical Direction	105.92	-

(h) Calculation of Forces Due to Wind - with Live Load

(h) 1. In Tansverse Direction

(i) Wind Force on Superstructure (As per Clause 209.3 of IRC:6-2010)

	LWL condition	HFL condition
Basic wind speed (V_b) (As per Fig.6)	40.89 m/s	42.73 m/s
H (avg. ht. of exposed surface from the ground level)	14.47 m	6.96 m
From table 4		
V_z (hourly mean speed of wind at height H)	29.05 m/s	27.80 m/s
P_z (horizontal wind pressure at height H)	507.28 N/m²	463.70 N/m²
Since table 4 is for basic wind speed (V_b) of 33 m/s		
V_z calculated	36.0 m/s	36.0 m/s
P_z calculated	779.0 N/m²	777.6 N/m²
In hilly areas due to funnelling, P_z is increased by 20%	779.0 N/m²	777.6 N/m²

As per Clause 209.3.3

Transverse wind force F_T (in N) = . $P_z \times A_1 \times G \times C_D$ **eq.1**

Here,

A_1 is the area of the superstructure as seen in elevation including the floor system and crash barrier

Height of 43m span including height of crash barrier, d	=	3.850	m
Height of 43m span including height of crash barrier, d	=	3.850	m
So, Area A_1 at which F_T is acting (43m Span)	=	3.850	m^2/m
So, Area A_1 at which F_T is acting (43m Span)	=	3.850	m^2/m
G is the gust factor (spans upto 150 m)	=	2	
C_D is the drag coefficient			
width of deck (b)	=	12	
b/d	=	3.16	
For bridge decks supported by single beam, C_D (For above b/d ratio)	=	1.44	
For bridge deck supported by two or more beams, C_D	=	1.44	

Putting all these values in eq.1

	LWL condition	HFL condition
F_T calculated 43m span	8651.11 N/m	8635.2 N/m
F_T For 43 m span	37.92 t	37.85 t
F_T For 43 m span	37.92 t	37.85 t
Long. Force on superstruture FL (25% of F_T) (As per Clause 209.3.4) (43m span)	9.48 t	9.46 t

Upward or downward vertical wind load F_V (in N) (As per Clause 209.3.5) = $P_Z \times A_3 \times G \times C_L$ **eq. 2**

Here,

So, Area A3 at which F_T is acting (43m Span)	=	12.15	m^2/m
So, Area A3 at which F_T is acting (43m Span)	=	12.15	m^2/m
C_L is the lift coefficient	=	0.75	
Other parameters P_Z and G remains same as used above			

	LWL condition	HFL condition
F_V(+/-) 43m Span	14197.787 N/m	14171.6 N/m
43m Span	14197.787 N/m	14171.6 N/m
F_v (For 43m span)	62.23 t	62.12 t

(ii) Wind Force on Live Load

Transverse wind force F_T (in N) = $P_Z \times A_3 \times G \times C_D$ **eq. 3**

(acting at a ht. of 3m above roadway)

Height of crash barrier above roadway = 1.10 m
Net ht. at which F_T is acting = 1.90 m
So Area (L x ht. at which F_T is acting) = 1.90 m^2/m
Drag coefficient, C_D = 1.20 (As per Clause 209.3.6)

	LWL condition	HFL condition
Putting these values in eq.3, F_T calculated	3552.37 N/m	3545.8 N/m
	0.36 t/m	0.36 t/m
F_T (For 43 m span)	15.57 t	15.54 t
F_T (For 43 m span)	15.57 t	15.54 t
Long. Force on superstruture FL (25% of F_T) (As per Clause 209.3.4) (43m span)	3.89 t	3.89 t

(iii) Wind Force on Substructure *(As per Clause 209.4 of IRC:6-2010)*

F_T on pier and pier cap is calculated in the same way as calculated in wind forces on superstructure

	LWL condition	HFL condition
For pier,		
Dia. of pier	2.25 m	2.25 m
Size of pier	0.00 m	0.00 m
Clear ht. of pier	6.57 m	0.56 m
A_1	14.77 m^2	1.27 m^2

C_D [For height / breadth ratio of 40] 0.60 (As per table 6)

So, F_T on pier	13809 N 1:41 t	1186.22 N 0.12 t
F_L on pier	0.35 t	0.03 t

	LWL condition	HFL condition
For pier cap,		
Width of pier cap	3.65 m	3.65 m
Depth of pier cap	2.00 m	2.00 m
A_1	7.30 m^2	7.30 m^2

t/b = 1.61 (As per table 6)

So, F_T on pier cap	18354 N 1.87 t	18320.1 N 1.87 t
F_L on pier cap	0:47 t	0.47 t

(h) 2. in Longitudinal Direction

With POT-PTFE		LWL condition	HFL condition
Wind force on superstructure		13.37 t	13.35 t
Total long. Force		13.37 t	13.35 t
	$\mu 1$	0.05	
	$\mu 2$	0.03	
Frictional force on free bearing	$\mu 2(Rg)$	14.55 t	14.55 t
Frictional force on fixed bearing	F_h-$\mu 2(Rg)$	-1.18 t	-1.18 t
	$F_h/2+\mu 1(Rg+Rq)$	36.54 t	36.54 t
	F_h	13.37 t	13.35 t

** Force on free bearing is acting opposite to the force on fixed bearing. hence ignored to be on conservative side.*

Thus, max force on a pier	**36.54 t**	**36.54 t**

	LWL condition			HFL condition		
LONGITUDINAL WIND FORCE	Force (t)	Lever arm (m)	Moment (tm)	Force (t)	Lever arm (m)	Moment (tm)
Horizontal Force	36.54	9.07	331.25	36.54	9.07	331.25
on Pier cap	0.47	7.57	3.54	0.47	7.57	3.53
on Pier	0.35	3.28	1.16	0.03	6.28	0.19
Total longitudinal Wind force	**37.36**		**335.94**	**37.04**		**334.97**

	LWL condition			HFL condition		
TRANSVERSE WIND FORCE DUE TO	Force (t)	Lever arm (m)	Moment (tm)	Force (t)	Lever arm (m)	Moment (tm)
Live Load	15.57	13.92	216.67	15.54	13.92	216.27
Superstructure (43 m span)	18.96	11.04	209.32	18.93	11.04	208.93
Superstructure (43 m span)	18.96	11.04	209.32	18.93	11.04	208.93
Pier cap	1.87	7.57	14.15	1.87	7.57	14.13
Pier	1.41	3.28	4.62	0.12	6.28	0.76
Total transverse Wind force	**41.20**		**654.08**	**39.84**		**649.03**

(h) 3. Summary of Wind Forces & its Moment

LWL condition	Force (t)	Moment (tm)	HFL condition	Force (t)	Moment (tm)
Transverse Direction	41.20	654.08	Transverse Direction	39.84	649.03
Longitudinal Direction	37.36	335.94	Longitudinal Direction	37.04	334.97
Vertical Direction (+/-)	62.23	-	Vertical Direction (+/-)	62.12	-

2. SUMMARY OF FORCES AT PIER BASE (THREE LANE TRAFFIC)

LOAD	V (Ton)	HL (Ton)	HT (Ton)	Torsion	MT (t-m)	ML (t-m)
DL (including pier and pier cap)	965.3	0.0	0.0	0.0	189.7	0.0
SIDL	70.0	0.0	0.0	0.0	16.6	0.0
SURFACING	100.0	0.0	0.0	0.0	23.7	0.0
FPLL	0.0	0.0	0.0	0.0	0.0	0.0
HORIZONTAL FORCE (NORMAL CONDITION)	0.0	39.7	0.0	0.0	0.0	359.7
SEIS. TRANS	0.0	0.0	128.8	0.0	1336.2	0.0
SEIS. LONG	0.0	154.4	0.0	0.0	0.0	1329.4
SEIS. VERT	105.9	0.0	0.0	0.0	0.0	0.0
WIND WITHOUT LL (LWL)	82.2	37.7	54.4	0.0	577.8	338.1
WIND WITHOUT LL(HFL)	75.1	37.2	48.2	0.0	523.5	336.4
WIND LL	62.2	37.4	41.2	0.0	654.1	335.9
LL (Max V / MAX MT Case)	165.0	0.0	12.8	0.0	342.4	77.3
LL (Max ML Case)	140.8	0.0	10.9	0.0	463.7	78.9
TEMP.	0.0	0.0	0.0	0.0	0.0	0.0
BARGE IMPACT	0.0	0.0	688.8	0.0	6095.9	0.0
DIFF. SETTLEMENT	0.0	0.0	0.0	0.0	0.0	0.0
PRESTRESS EFFECTS	0.0	0.0	0.0	0.0	0.0	0.0
SHRINKAGE & CREEP	0.0	0.0	0.0	0.0	0.0	0.0

3. GEOTECHNICAL DESIGN OF FOUNDATION USING WORKING STRSS METHOD

3.1 Coefficient of Earth Pressure

As Per Coulomb's Theory, Coefficient of Active Earth Pressure is

$$Ka = \frac{SIN^2(\alpha+\phi)}{SIN^2\alpha.\ SIN(\alpha-\delta)\left[1+\sqrt{\frac{SIN(\phi+\delta).SIN(\phi-\beta)}{SIN(\alpha-\delta).SIN(\alpha+\beta)}}\right]}$$

Where

Φ = Angle of Internal Friction of Earth
α = Angle of Inclination of Back of Wall
δ = Angle of Internal Friction between Wall & Earth
β = Angle of Inclination of Backfill

Here

Φ =	33 °	=	0.576 Radian
α =	90 °	=	1.571 Radian
δ =	22.00 °	=	0.384 Radian

β =	0 °	=	0.000	Radian
		K_a =	0.2645	
		K_p =	8.084	
Therefore, Horizontal coefficient of Active earth pressure	= $K_a COS\delta$ =	K_{ha} =	0.2452	

3.2 Design of Circular Well Foundation

3.2.1 *Basic Design Data*

C/C Distance of Piers	43.00	m
FRL	74.37	m
Ground/Bed Level	66.50	m
Founding Level	34.00	m
Highest Flood Level	68.50	m
Low Water Level	61.00	m
Maximum Scour Level	53.00	m
For Combinations with Seismic Forces		
Scour Level Under Mean Flood (as per IRC:78-2014)	54.55	m
Scour Level Under L.W.L. (as per IRC:78-2014)	56.10	m
Depth of Superstructure (@ support location)	2.750	m
Thickness of Wearing Coat	0.050	m
Bearing Level	71.57	m
CG of Bearing Pad	71.97	m
Pier Cap Top Level	71.07	m
Depth of Pier Cap	2.00	m
Bottom of Pier Cap	69.07	m
Top of Pier	69.07	m
Top of Well Cap Level	62.50	m
Dia. of Pier	2.25	m
Width of Pier for Water Current Forces	2.25	m
Eccentricity of Pier W.R.T. Well in Transverse Dir	0.000	m
Eccentricity of Pier W.R.T. Well in Longitudinal Dir	0.000	m

3.2.2 *Well Dimensions*

Level, Where Dia of Well Changes (Above M.S.L.)	54.55	m
Diameter of Well (Above M.S.L.)	6.00	m
Diameter of Well (Below M.S.L.)	6.00	m

Well Cap Thickness	1.50	m
Projection of Curb Outside Steining	0.075	m
Sinking Depth, L	28.50	m
Steining Thickness Required	0.961	m
Steining Thickness Provided	1.000	m
Angle of Curb	36.55	degree
Height of Curb	1.750	m
Net Downward Load of Well (HFL Case)	770.0	t
Net Downward Load of Well (LWL Case)	1031.8	t

3.3 Load from Superstructure

	MAX M_L CASE	MAX. LOAD CASE
DL, SIDL & Substr.	1035.3	1035.3
Wearing Coat	100.0	100.0
DL, SIDL, WC & Substr.	1135.3	1135.3
Live Load (including FPLL)		
Vertical Reaction (t)	140.8	165.0
Long Moment due to DL, SIDL, WC & Substr. (t-m)	0.0	0.0
Long Moment due LL & FPLL (t-m)	78.9	77.3
Total Longitudinal Moment (t-m)	78.9	77.3
Trans. Moment due to DL, SIDL MT (t-m)	206.3	206.3
Trans. Moment due to WC MT (t-m)	23.7	23.7
Trans. Moment due to LL & FPLL MT (t-m)	774.8	706.9
Braking Tractive Force F_h (t)	22.8	22.8
Horizontal Force	39.7	39.7
Moment In Long. Direc., ML (due to F_h) (t-m)	1490.8	1490.8
Horizontal Force due to barge impact (t)	688.8	688.8
Transverse Moment at founding level (t-m)	25726.9	25726.9

3.4 Water Current Forces (HFL Case)

V^2 at Surface	15.905
Height of Water above M.S.L	15.500 m
V^2 at Bottom of Pier Cap & Top of Pier	0.000
V^2 at Top of Well Cap	9.748
Total Horiz. Force Due to Water Current (Trans Dir.)	**15.48 t**
Total Moment at Found. Level (Trans Dir.)	**430.17 t-m**

Seismic Case with Mean Water Current Forces (Mean Flood Case)	
Mean Flood Level (MFL) =	68.50 m
(MFL is Assumed at HFL, on Conservative Side)	
V^2 at Surface	15.90
Height of Water above M.S.L	13.95 m
V^2 at Bottom of Pier Cap & Top of Pier	0.000
V^2 at Top of Well Cap	9.06
Total Horiz. Force Due to Water Current (Trans Dir.)	**13.20 t**
Total Moment at Found. Level (Trans Dir.)	**375.58 tm**

3.5 Seismic Forces

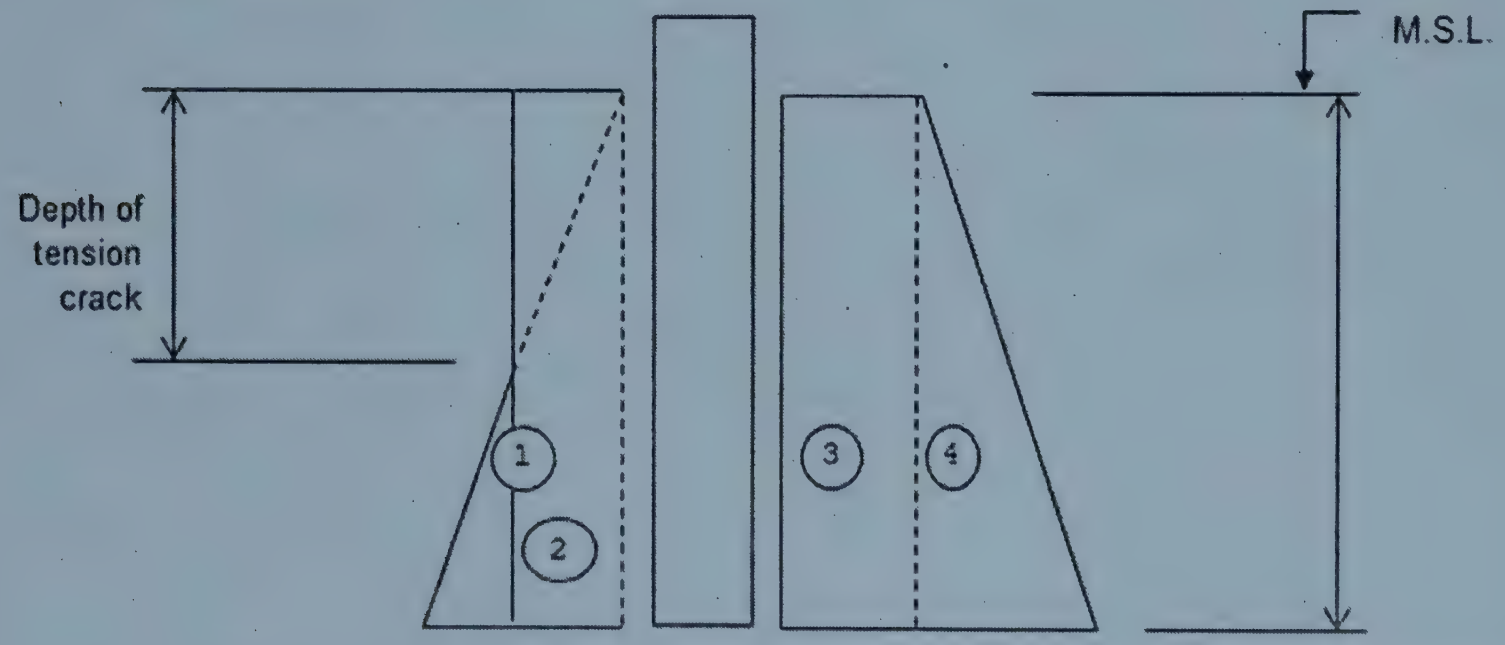
3.5.1 *In Longitudinal Direction (100%L + 30%T + 30%V)*

Horizontal Seismic Coefficient in Longitudinal Direction, a_h	0.136
Scour Level in L.W.L. Case (0.9 Times the MSL)	54.55 m
Considering LL in Seismic Case	20%
Longitudinal Seismic Force	**Service Condi.**
Braking Force	4.554 t
on Superstructure & Substructure	154.40 t
on Well Cap	14.42 t
on Well Steining (Upto MSL)	34.45 t
on Well Steining L.W.L. Case	26.17 t
on Water Inside Steining	8.89 t
on Sandfill Inside Steining	0.00 t
on Top Plug	0.00 t
Moment at Founding Level Due to Seis. Force on	
Braking Force	201.7 t-m
Superstructure & Substructure	5729.7 t-m
Well Cap	400.2 t-m
Steining	819.0 t-m
Water Inside Steining	211.3 t-m
Total Hydrodynamic Force	33.89 t
its Moment at Founding Level	869.74 tm
Total Force in Longitudinal Dir. (Including Braking & Hydrodynamic Force)	250.6 t
30% Transverse Seismic Force	62.0 t
Total Moment at Founding Level in Longitudinal Direction	8231.6 t-m
Moment at Founding Level Due to Seismic in Trans. Direction	2109.4 t-m
Increase in Vertical Load	41.6 t

3.5.2 *in Transverse Direction (100%T + 30%L + 30%V)*

Horizontal Seismic Coefficient in Transverse Direction, a_h	. 0.110
Considering Seismic Force on LL in Trans Direction on	20%
Maximum Live Load Reaction	33.0 t
Seismic Force on LL	3.64 t
Trans. Seismic Force on Superstr. & Substr.	128.82 t
Well Cap	11.69 t
Steining	27.93 t
Water Inside Steining	7.21 t
Total Hydrodynamic Force	27.48 t
its Moment at Founding Level	705.19 tm
Total Force in Transverse Dir. (Including Hydrodynamic Force)	**206.8 t**
30% Longitudinal Seismic Force (Including 20% Braking & Hydrodynamic Force)	**78.4 t**
Total Moment at Founding Level in Transverse Direction	**7031.5 tm**
Moment at Founding Level Due to Seismic in Long. Direction (Including Braking & Hydrodynamic Force)	**2610.7 tm**
Increase in Vertical Load	**41.6 t**

3.6 Passive Relief



Depth of Tension Crack ($2c/\gamma\ Ka^{0.5}$)	0.0 m
Grip Length below M.S.L. in Normal Case	19.00 m
Grip Length below M.S.L. in Seismic Case	20.55 m
Coefficient factor for circular well for passive relief	1
Coefficient of Active Earth Pressure, Ka for Normal Case	0.264
Coefficient of Active Earth Pressure, Ka for Seismic Case	0.264
Coefficient of Passive Earth Pressure, Kp	8.084

Normal Case		
Height of Earth		19.00 m
F. O. S.		2.0
Net Forces and Moment of Lateral Resistance ($0.5g_{sub}$(Kp-Ka)cosd.h^2.D/F.O.S.)	Force (t)	Moment (tm)
Rectangle component due to Active Earth Pressure (2)	0.0	0.00
Rectangle component due to Passive Earth Pressure (3)	0.0	0.00
Tri. Comp. due to AEP (1) & PEP (4) {(4)-(1)}	3926.1	24865.3
	3926.1	24865.3
Moment due to Vertical Component of Active & Passive Earth Pressure		0.0
Net Force of Lateral Resistance		3926.1 t
Moment Due to Passive Relief (PR)		24865.3 tm
Seismic Case		
Height of Earth		20.550 m
F. O. S.		1.6
Net Forces and Moment of Lateral Resistance ($0.5g_{sub}$(Kp-Ka)cosd.h^2.D/F.O.S.)	Force (t)	Moment (tm)
Rectangle component due to Active Earth Pressure (2)	0.0	0.00
Rectangle component due to Passive Earth Pressure (3)	0.0	0.00
Tri. Comp. due to AEP (1) & PEP (4) {(4)-(1)}	5741.0	39325.9
	5741.0	39325.9
Moment due to Vertical Component of Active & Passive Earth Pressure		0.0
Net Force of Lateral Resistance		5741.0 t
Moment Due to Passive Relief (PR)		39325.9 tm

3.7 Geo-Technical Design of Foundations using Working stress method-IRC:78

	LC-1	LC-2	LC-3
	Basic Combination	Seismic	Accidental
Loads	Wind + LL	-	-
DL & SIDL	1	1	1
Surfacing	1	1	1
Live load	1	0.2	1
Seismic force	-	1	-
Water Current	1	1	-
Wind	1	-	-
Thermal	-		1
Settlement	1	1	-

Barge Impact	-	-	1
Prestress Effects	-	-	-

3.8 Moment Due to Tilt and Shift

TILT (1 IN 80) 0.356 m

SHIFT (150mm) 0.150 m

DESCRIPTION	(MAX. LD & LWL CASE CONSIDERED)
W1=Net Dnward LD on Top of Well	1300.3 t
Moment about C.L. of Well (Due to W1)	658.3 tm
W2=Net Downward Load of Well	1031.8 t
Total Moment about C.L. of Well (Due to W2)	183.8 tm
Total Moment Due to Tilt and Shift	**842.0** tm

3.9 Summary of Loads & Moments

3.9.1 *HFL Wind Case (LC-1)*

		MAX M_L CASE	MAX. LD CASE
Downward Load	t	2108.3	2132.5
Moment in Longitudinal Direction	t-m	1479.6	1478.0
Moment in Transverse Direction	t-m	3263.2	3195.4
Resulant Mom. (M_R) (including tilt & shift)	t-m	4456.5	4394.2
Passive Relief (P_R)	t-m	31081.6	31081.6
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m^2	71.0	71.8
Minimum Base Pressure	t/m^2	71.0	71.8

3.9.2 *LWL Wind Case (LC-1)*

		MAX M_L CASE	MAX. LD CASE
Downward Load	t	2370.1	2394.3
Moment in Longitudinal Direction	t-m	1479.6	1478.0
Moment in Transverse Direction	t-m	2833.0	2765.2
Resulant Mom. (M_R) (including tilt & shift)	t-m	4069.7	4009.0
Passive Relief (P_R)	t-m	31081.6	31081.6
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m^2	79.8	80.6
Minimum Base Pressure	t/m^2	79.8	80.6

As plastic moment is dependent on sectional capacity, it is assumed that section is designed for exact requirement. Hence the seismic moment and shear at the base is multiplied by overstrength factor to simplify the process.

Factored loads after multiplying over strength factor of 1.35

3.9.3 *Seismic Longitudinal With LWL Case (LC-2)*

		MAX M_L CASE	MAX. LD CASE
Downward Load (with 20% live load)	t	2236.7	2241.6
Moment in Longitudinal Direction	t-m	11128.5	11128.1
Moment in Transverse Direction	t-m	2494.4	2480.8
Resulant Mom. (M_R) (including tilt & shift)	t-m	12200.8	12197.6
Passive Relief (P_R)	t-m	39325.9	39325.9
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	75.3	75.5
Minimum Base Pressure	t/m²	75.3	75.5

3.9.4 *Seismic Longitudinal With Mean Flood Level (LC-2)*

		MAX M_L CASE	MAX. LD CASE
Downward Load (with 20% live load)	t	1975.0	1979.8
Moment In Longitudinal Direction	t-m	11128.5	11128.1
Moment In Transverse Direction	t-m	2870.0	2856.4
Resulant Mom. (M_R) (including tilt & shift)	t-m	12288.8	12285.1
Passive Relief (P_R)	t-m	39325.9	39325.9
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	66.5	66.6
Minimum Base Pressure	t/m²	66.5	66.6

3.9.5 *Seismic Transverse With LWL Case (LC-2)*

		MAX M_L CASE	MAX. LD CASE
Downward Load (with 20% live load)	t	2236.7	2241.6
Moment in Longitudinal Direction	t-m	2626.5	2626.2
Moment in Transverse Direction	t-m	9877.5	9863.9
Resulant Mom. (M_R) (including tilt & shift)	t-m	11017.0	11003.8
Passive Relief (P_R)	t-m	39325.9	39325.9
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	75.3	75.5
Minimum Base Pressure	t/m²	75.3	75.5

3.9.6 *Seismic Transverse With Mean Flood Level (LC-2)*

		MAX M_L CASE	MAX. LD CASE
Downward Load (with 20% live load)	t	1975.0	1979.8
Moment in Longitudinal Direction	t-m	2626.5	2626.2
Moment in Transverse Direction	t-m	10253.1	10239.5
Resulant Mom. (M_R) (including tilt & shift)	t-m	11380.4	11367.1
Passive Relief (P_R)	t-m	39325.9	39325.9
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	66.5	66.6
Minimum Base Pressure	t/m²	66.5	66.6

3.9.7 *Barge Impact With HFL (LC-3)*

		MAX M_L CASE	MAX. LD CASE
Downward Load (with 100% live load)	t	2046.1	2070.3
Moment in Longitudinal Direction	t-m	1569.6	1568.0
Moment in Transverse Direction	t-m	27107.3	27039.5
Resulant Mom. (M_R) (including tilt & shift)	t-m	27994.8	27926.9
Passive Relief (P_R)	t-m	39325.9	39325.9
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	68.9	69.7
Minimum Base Pressure	t/m²	68.9	69.7

3.10 Summary of Base Pressure

Absolute Maximum in Wind Case	80.6	t/m²	<	89.00	Hence Safe
Absolute Minimum in Wind Case	71.0	t/m²	>	0.00	Hence Safe
Absolute Maximum in Seismic Case	75.5	t/m²	<	89.00	Hence Safe
Absolute Minimum in Seismic Case	66.5	t/m²	>	0.00	Hence Safe
Absolute Maximum in Barge Impact Case	69.7	t/m²	<	89.00	Hence Safe
Absolute Minimum in Barge Impact Case	68.9	t/m²	>	0.00	Hence Safe

4. GEOTECHNICAL DESIGN OF FOUNDATION USING LIMIT STATE METHOD

4.1 Coefficient of Earth Pressure (For Set 1)

As per Coulomb's theory, coefficient of active earth pressure is

$$Ka = \frac{SIN^2(\alpha+\phi)}{SIN^2\alpha.\ SIN(\alpha-\delta)\left[1+\sqrt{\frac{SIN(\phi+\delta).SIN(\phi-\beta)}{SIN(\alpha-\delta).SIN(\alpha+\beta)}}\right]^2}$$

Where

Φ = Angle of Internal Friction of Earth

α = Angle of Inclination of Back of Wall

δ = Angle of Internal Friction Between Wall & Earth

β = Angle of Inclination of Backfill

Here

Φ =	33°	=	0.576	Radian
α =	90°	=	1.571	Radian
δ =	22.00°	=	0.384	Radian
β=	0°	=	0.000	Radian
		K_a =	0.2645	
		K_p =	8.084	
Therefore, Horizontal coefficient of Active earth pressure	$K_aCOS\delta$ =	K_{ha} =	0.2452	
		K_{pa} =	7.4956	

4.2 Coefficient of Earth Pressure (For Set 2)

As per Coulomb's theory, coefficient of active earth pressure is

$$Ka = \frac{SIN^2(\alpha+\phi)}{SIN^2\alpha.\ SIN(\alpha-\delta)\left[1+\sqrt{\frac{SIN(\phi+\delta).SIN(\phi-\beta)}{SIN(\alpha-\delta).SIN(\alpha+\beta)}}\right]^2}$$

Where

Φ = Angle of Internal Friction of Earth

α = Angle of Inclination of Back of Wall

δ = Angle of Internal Friction Between Wall & Earth

β = Angle of Inclination of Backfill

Here,

Φ =	27.45°	=	0.479	Radian
α =	90°	=	1.571	Radian
δ =	18.30°	=	0.319	Radian
β=	0°	=	0.000	Radian
		K_a =	0.3282	

		K_p =	4.929
Therefore, Horizontal coefficient of Active earth pressure	$K_aCOS\delta$ =	K_{ha} =	0.3116
		K_{pa} =	4.6797

4.3 Calculation of passive resistance for limit state design of well

Angle of Shearing Resistance (φ) =	33	°	
Submerged density of soil =	1	t/m³	
Outer diameter of Well =	6	m	
Embedded length of Well =	19	m	(Normal Case)
Embedded length of Well =	20.6	m	(Seismic Case)

As per Table 2, (of Limit Sate Design of Open Foundation)

Design angle of shearing Resistance, $\varphi^1 = \tan^{-1}((\tan\varphi)/\gamma_m)$

For Set 1, γ_m =	1	
Therefore φ^1=	33	°
For Set 2, γ_m =	1.25	
Therefore φ^1=	27.45	°

Now, for calculating the Passive Resistance as per Table B4 of IRC 6:2017, Set 1 factors are used for LC-1, LC-2 and Set 2 factors are used for LC-3, LC-4, LC-5, LC-6.

FOR LC-1, LC-2

K_{ha} =	0.245		
K_{hp} =	7.496		
Force of Passive Resistance =	7719.4	t	0.5 x 1 (7.496 - 0.245 x 1.5) x 19 x 19 x 6
Total Resisting Moment =	48889.7	tm	
Allowable Resisting Moment =	48889.7	tm	

FOR LC-3, LC-4

K_{ha} =	0.312		
K_{hp} =	4.680		
Force of Passive Resistance =	3869.3	t	0.5x1(4.68x0.85-0.312x1.3)x19x19x6
Total Resisting Moment =	24505.5	tm	
Allowable Resisting Moment =	24505.5	tm	

FOR LC-5A, 5B

K_{ha} =	0.312		
K_{hp} =	4.680		
Force of Passive Resistance =	5534.1	t	0.5 x 1 (4.68 - 0.312) x 20.55 x 20.55 x 6
Total Resisting Moment =	37908.4	tm	
Allowable Resisting Moment =	37908.4	tm	

FOR LC-6

K_{ha} =	0.312		
K_{hp} =	4.680		
Force of Passive Resistance =	4730.7	t	0.5 x 1 (4.68 - 0.312) x 19 x 19 x 6
Total Resisting Moment =	29961.3	tm	
Allowable Resisting Moment =	29961.3	tm	

Load Definition:

LC-1	L.L. Leading +Wind Load Acc. _LWL/HFL Case	(Action Combination 1)
LC-2	L.L. Acc +Wind Load Leading _LWL/HFL Case	
LC-3	L.L. Leading +Wind Load Acc. _LWL/HFL Case	(Action Combination 2)
LC-4	L.L. Acc +Wind Load Leading _LWL/HFL Case	
LC-5A	Seismic Long. + 30% Seismic Trans. + 30% Seismic Vert (+v)_LWL/HFL Case	
LC-5B	30% Seismic Long. + Seismic Trans. + 30% Seismic Vert (+v)_LWL/HFL Case	
LC-6	Accidental Combination	

4.4 Design of Circular Well Foundation

4.4.1 *Basic design data*

C/C Distance of Piers	43.00 m
FRL	74.37 m
Ground/Bed Level	66.50 m
Founding Level	34.00 m
Highest Flood Level	68.50 m
Low Water Level	61.00 m
Maximum Scour Level	53.00 m
For combinations with seismic forces	
Scour Level Under Mean Flood (as per IRC:78)	54.55 m
Scour Level Under L.W.L. (as per IRC:78)	56.10 m
Depth of Superstructure (@ support location)	2.750 m
Thickness of Wearing Coat	0.050 m
Bearing Level	71.57 m
CG of Bearing Pad	71.97 m
Pier Cap Top Level	71.07 m
Depth of Pier Cap	2.00 m
Bottom of Pier Cap	69.07 m
Top of Pier	69.07 m
Top of Well Cap Level	62.50 m

Dia. of Pier		2.25	m
Width of Pier for Water Current Forces		2.25	m
Eccentricity of Pier w.r.t. Well in Transverse Dir		0.000	m
Eccentricity of Pier w.r.t. Well in Longitudinal Dir		0.000	m

4.4.2 *Well dimensions*

Level, where Dia of Well Changes (Above M.S.L.)		54.55	m
Diameter of Well (Above M.S.L.)		6.00	m
Diameter of Well (Below M.S.L.)		6.00	m
Well Cap Thickness		1.50	m
Projection of Curb Outside Steining		0.075	m
Horizontal Seismic Coefficient, a_h X 1.35		0.136	Longitudinal Direction
		0.110	Transverse Direction
Max. Surface Velocity Of Water, V (Design Velocity)	2.82 x √2	3.99	m/sec
Sinking Depth, L		28.50	m
Steining Thickness Required		0.961	m
Steining Thickness Provided		1.000	m
Angle of Curb		36.55	degree
Height of Curb		1.750	m
Dead Load of Well			
Net Downward Load of Well (HFL Case)		770.0	t
Net Downward Load of Well (LWL Case)		1031.8	t

4.5 Load From Superstructure

	MAX M_L CASE	MAX. LOAD CASE
DL, SIDL & SUBSTR.	1035.3	1035.3
Wearing Coat	100.0	100.0
DL, SIDL, WC & SUBSTR.	1135.3	1135.3
LIVE LOAD (including FPLL)		
Vertical Reaction (t)	140.8	165.0
Long Moment due to DL, SIDL, WC & Substr. (t-m)	0.0	0.0
Long Moment due LL & FPLL (t-m)	78.9	77.3
Total Longitudinal Moment (t-m)	78.9	77.3
Trans. Moment due to DL, SIDL M_T (t-m)	206.3	206.3

Trans. Moment due to WC M_T (t-m)	23.7	23.7
Trans. Moment due to LL & FPLL M_T (t-m)	774.8	706.9
Braking Tractive Force F_h (t)	22.8	22.8
Horizontal Force	39.7	39.7
Moment In Long. Direc., M_L (due to F_h) (t-m)	1490.8	1490.8
Horizontal Force due to barge impact (t)	688.8	688.8
Transverse Moment at founding level (t-m)	25726.9	25726.9

4.6 Water Current Forces (HFL Case)

V^2 at Surface	15.905
Height of Water above M.S.L	15.500 m
V^2 at Bottom of Pier Cap & Top of Pier	0.000
V^2 at Top of Well Cap	9.748
Total Horiz. Force Due to Water Current (Trans Dir.)	**15.48 t**
Total Moment at Found. Level (Trans Dir.)	**430.17 t-m**
Seismic Case With Mean Water Current Forces (Mean Flood Case)	
Mean Flood Level (MFL) =	68.50 m
(MFL is Assumed at HFL, on Conservative Side)	
V^2 at Surface	15.90
Height of Water above M.S.L	13.95 m
V^2 at Bottom of Pier Cap & Top of Pier	0.000
V^2 at Top of Well Cap	9.06
Total Horiz. Force Due to Water Current (Trans Dir.)	**13.20 t**
Total Moment at Found. Level (Trans Dir.)	**375.58 tm**

4.7 Seismic Forces

4.7.1 *In Longitudinal Direction (100%L + 30%T + 30%V)*

Horizontal Seismic Coefficient In Longitudinal Direction, a_h	0.136
Scour Level in L.W.L. Case (0.9 Times The MSL)	54.55 m
Considering LL in Seismic Case	20%
Longitudinal Seismic Force	**SERVICE CONDI.**
Braking Force	4.554 t
On Superstructure & Substructure	154.40 t
On Well Cap	14.42 t
On Well Steining (Upto MSL)	34.45 t
On Well Steining L.W.L. Case	26.17 t
On Water Inside Steining	8.89 t

Moment at Founding Level due to Seis. Force on	
Braking Force	201.7 t-m
Superstructure & Substructure	5729.7 t-m
Well Cap	400.2 t-m
Steining	819.0 t-m
Water Inside Steining	211.3 t-m
Total Hydrodynamic Force	33.89 t
Its Moment at Founding Level	869.74 tm
Total Force in Longitudinal Dir. (including braking & hydrodynamic force)	250.6 t
30% Transverse Seismic Force	62.0 t
Total Moment at Founding Level in Longitudinal Direction	8231.6 t-m
Moment at Founding Level Due to Seismic in Trans. Direction	2109.4 t-m
Increase in Vertical Load	41.6 t

4.7.2 *In Transverse Direction (100%T + 30%L + 30%V)*

Horizontal Seismic Coefficient in Transverse Direction, a_h	0.110	
Seismic Force on LL	3.64	t
Trans. Seismic Force on Superstr. & Substr.	128.82	t
Well Cap	11.69	t
Steining	27.93	t
Water Inside Steining	7.21	t
Moment at Founding Level Due to Seis. Force on LL	158.8	t-m
Superstructure & Substr.	5007.7	tm
Well Cap	324.4	tm
Steining	664.0	tm
Water Inside Steining	171.3	tm
Total Hydrodynamic Force	27.48	t
Its Moment at Founding Level	705.19	tm
Total Force in Transverse Dir. (Including Hydrodynamic Force)	206.8	t
30% Longitudinal Seismic Force (Including 20% Braking & Hydrodynamic Force)	78.4	t
Total Moment at Founding Level in Transverse Direction	7031.5	tm
Moment at Founding Level Due to Seismic in Long. Direction (Including Braking & Hydrodynamic Force)	2610.7	tm
Increase in Vertical Load	41.6	t
Base Area	29.706	m2
Section Modulus	22.836	m3

4.8 Geo-Technical Design of Foundations using Limit State-

	Table B.4						Table B.3
	LC-1	LC-2	LC-3	LC-4	LC-5A, 5B	LC-6	LC-7
	Combination-1		Combination-2		Seismic	Accidental	Quasi
Loads	LL Lead	Wind Lead	LL Lead	Wind Lead	-	-	-
DL & SIDL	1.35	1.35	1	1	1.35	1	1
Surfacing	1.75	1.75	1	1	1.75	1	1.2
Live load	1.5	1.15	1.3	1	0.2	0.75	-
Seismic force	-	-	-	-	1.5	-	-
Water Current	1	1	1	1	1	-	-
Wind	0.9	1.5	0.8	1.3	-	-	-
Thermal	-					0.5	0.5
Settlement	1	1	1	1	1	-	1
Barge Impact	-	-	-	-	-	1	-
Prestress Effects	-	-	-	-	-	-	-

(IRC: 6-2017)

4.9 Moment Due to Tilt and Shift

Tilt (1 In 80) 0.356 m

Shift (150mm) 0.150 m

Description	**(Max. LD & LWL Case Considered)**				**(Factored loads)**
	For LC-1, LC-2	For LC-3, LC-4	For LC-5	For LC-5	For LC-5
W1=Net Dnward LD on Top of Well	1820.1	1349.8	1605.6	1259.0	1155.3 t
Moment about C.L. of Well (Due to W1)	921.4	683.3	812.8	637.4	584.8 tm
W2=Net Downward Load of Well	1392.9	1031.78	1392.90	1031.78	1031.78 t
Total Moment about C.L. of Well (Due to W2)	248.1	183.8	248.1	183.8	183.8 tm
Total Moment Due to Tilt and Shift	**1169.5**	**867.1**	**1060.9**	**821.2**	**768.6** tm

4.10 Summary of Loads & Moments

4.10.1 *Load Combination 1*

(a) HFL Wind Case (LC-2) *with wind leading*		**MAX M_L CASE**	**MAX. LD CASE**
Downward Load	t	2867.4	2895.2
Moment in Longitudinal Direction	t-m	2191.8	2190.0
Moment in Transverse Direction	t-m	4383.6	4305.5
Resulant Mom. (M_R) (including tilt & shift)	t-m	6117.8	6047.3

Passive Relief (P_R)	t-m	48889.7	48889.7
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	96.5	97.5
Minimum Base Pressure	t/m²	96.5	97.5

(b) LWL Wind Case (LC-2) *with wind leading*		**MAX M_L CASE**	**MAX. LD CASE**
Downward Load	t	3220.8	3248.6
Moment in Longitudinal Direction	t-m	2191.8	2190.0
Moment in Transverse Direction	t-m	3939.2	3875.3
Resulant Mom. (M_R) (including tilt & shift)	t-m	5724.7	5668.1
Passive Relief (P_R)	t-m	48889.7	48889.7
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	108.4	109.4
Minimum Base Pressure	t/m²	108.4	109.4

(C) HFL Wind Case (LC-1) *with live load leading*		**MAX M_L CASE**	**MAX. LD CASE**
Downward Load	t	2879.3	2915.6
Moment in Longitudinal Direction	t-m	1379.0	1376.6
Moment in Transverse Direction	t-m	3557.8	3456.0
Resulant Mom. (M_R) (including tilt & shift)	t-m	5013.6	4917.9
Passive Relief (P_R)	t-m	48889.7	48889.7
Net Moment (M_R - P_R)		0.0	0.0
Maximum Base Pressure	t/m²	96.9	98.1
Minimum Base Pressure	t/m²	96.9	98.1

(d) LWL Wind Case (LC-1) *with live load leading*		**MAX M_L CASE**	**MAX. LD CASE**
Downward Load	t	3232.7	3269.0
Moment in Longitudinal Direction	t-m	1379.0	1376.6
Moment in Transverse Direction	t-m	3127.6	3025.8
Resulant Mom. (M_R) (including tilt & shift)	t-m	4616.0	4522.1
Passive Relief (P_R)	t-m	48889.7	48889.7
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	108.8	110.0
Minimum Base Pressure	t/m²	108.8	110.0

4.10.2 *Load Combination 2*

(a) HFL Wind Case (LC-4) *with wind leading*		**MAX M_L CASE**	**MAX. LD CASE**
Downward Load	t	2127.0	2151.2
Moment in Longitudinal Direction	t-m	1899.8	1898.2
Moment in Transverse Direction	t-m	3811.7	3743.8
Resulant Mom. (M_R) (including tilt & shift)	t-m	5809.0	5738.5
Passive Relief (P_R)	t-m	24505.5	24505.5
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	71.6	72.4
Minimum Base Pressure	t/m²	71.6	72.4

(b) LWL WIND CASE (LC-4) *with wind leading*		**MAX M_L CASE**	**MAX. LD CASE**
Downward Load	t	2388.7	2412.9
Moment In Longitudinal Direction	t-m	1899.8	1898.2
Moment In Transverse Direction	t-m	3381.5	3313.7
Resulant Mom. (M_R) (including tilt & shift)	t-m	5415.9	5359.4
Passive Relief (P_R)	t-m	24505.5	24505.5
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	80.4	81.2
Minimum Base Pressure	t/m²	80.4	81.2

(C) HFL WIND CASE (LC-3)		**MAX M_L CASE**	**MAX. LD CASE**
Downward Load	t	2138.1	2169.5
Moment in Longitudinal Direction	t-m	1223.1	1221.0
Moment in Transverse Direction	t-m	3312.8	3041.8
Resulant Mom. (M_R) (including tilt & shift)	t-m	4708.0	4612.4
Passive Relief (P_R)	t-m	24505.5	24505.5
Net Moment (M_R - P_R)		0.0	0.0
Maximum Base Pressure	t/m²	72.0	73.0
Minimum Base Pressure	t/m²	72.0	73.0

(d) LWL WIND CASE (LC-3) *with live load leading*		**MAX M_L CASE**	**MAX. LD CASE**
Downward Load	t	2399.9	2431.3
Moment in Longitudinal Direction	t-m	1223.1	1221.0
Moment in Transverse Direction	t-m	2699.8	2611.6
Resulant Mom. (M_R) (including tilt & shift)	t-m	4310.4	4216.5

Passive Relief (P_R)	t-m	24505.5	24505.5
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	80.8	81.8
Minimum Base Pressure	t/m²	80.8	81.8

As plastic moment is dependent on sectional capacity, it is assumed that section is designed for exact requirement. Hence the seismic moment and shear at the base is multiplied by overstrength factor to simplify the process.

Factored loads after multiplying over strength factor of 1.35

4.10.3 *Seismic Longitudinal With LWL Case (LC-5A)*

		MAX M_L CASE	MAX. LD CASE
Downward Load (with 20% live load)	t	3056.0	3060.8
Moment in Longitudinal Direction	t-m	16684.8	16684.5
Moment in Transverse Direction	t-m	3639.1	3625.5
Resulant Mom. (M_R) (including tilt & shift)	t-m	18169.6	18166.4
Passive Relief (P_R)	t-m	37908.4	37908.4
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	102.9	103.0
Minimum Base Pressure	t/m²	102.9	103.0

4.10.4 *Seismic Longitudinal with Mean Flood Level (LC-5A)*

		MAX M_L CASE	MAX. LD CASE
Downward Load (with 20% live load)	t	2702.6	2707.4
Moment in Longitudinal Direction	t-m	16684.8	16684.5
Moment in Transverse Direction	t-m	4014.7	4001.1
Resulant Mom. (M_R) (including tilt & shift)	t-m	18253.5	18250.0
Passive Relief (P_R)	t-m	37908.4	37908.4
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m²	91.0	91.1
Minimum Base Pressure	t/m²	91.0	91.1

4.10.5 *Seismic Transverse With LWL Case (LC-5B)*

		MAX M_L CASE	MAX. LD CASE
Downward Load (with 20% live load)	t	3056.0	3060.8
Moment in Longitudinal Direction	t-m	3931.8	3931.5
Moment in Transverse Direction	t-m	14713.7	14700.1
Resulant Mom. (M_R) (including tilt & shift)	t-m	16322.5	16309.3
Passive Relief (P_R)	t-m	37908.4	37908.4
Net Moment (M_R - P_R)	t-m	0.0	0.0

Maximum Base Pressure	t/m^2	102.9	103.0
Minimum Base Pressure	t/m^2	102.9	103.0

4.10.6 *Seismic Transverse With Mean Flood Level (LC-5B)*

		MAX M_L CASE	MAX. LD CASE
Downward Load (with 20% live load)	t	2702.6	2707.4
Moment in Longitudinal Direction	t-m	3931.8	3931.5
Moment In Transverse Direction	t-m	15089.3	15075.7
RESULANT MOM. (MR) (including tilt & shift)	t-m	16685.6	16672.4
PASSIVE RELIEF (PR)	t-m	37908.4	37908.4
NET MOMENT (MR - PR)	t-m	0.0	0.0
MAXIMUM BASE PRESSURE	t/m^2	91.0	91.1
MINIMUM BASE PRESSURE	t/m^2	91.0	91.1

4.10.7 *Barge Impact With HFL (LC-6)*

		MAX M_L CASE	MAX. LD CASE
Downward Load (with 100% live load)	t	2010.9	2029.0
Moment in Longitudinal Direction	t-m	1177.2	1176.0
Moment in Transverse Direction	t-m	26913.6	26862.7
Resulant Mom. (M_R) (including tilt & shift)	t-m	27760.5	27709.6
Passive Relief (P_R)	t-m	29961.3	29961.3
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m^2	67.7	68.3
Minimum Base Pressure	t/m^2	67.7	68.3

4.10.8 *SLS-Quasi Permanent LWL (LC-7)*

		MAX M_L CASE	MAX. LD CASE
Downward Load	t	2187.0	2187.0
Moment in Longitudinal Direction	t-m	57.8	57.8
Moment in Transverse Direction	t-m	230.0	230.0
Resulant Mom. (M_R) (including tilt & shift)	t-m	1005.8	1005.8
Passive Relief (P_R)	t-m	29961.3	29961.3
Net Moment (M_R - P_R)	t-m	0.0	0.0
Maximum Base Pressure	t/m^2	73.6	73.6
Minimum Base Pressure	t/m^2	73.6	73.6

4.11 Summary of Base Pressure

Absolute Maximum in Wind Case	**110.0**	**t/m²**	<	426.92	Hence Safe
Absolute Minimum in Wind Case	**71.6**	**t/m²**	>	0.00	Hence Safe
Absolute Maximum in Seismic Case	**103.0**	**t/m²**	<	469.61	Hence Safe
Absolute Minimum in Seismic Case	**91.0**	**t/m²**	>	0.00	Hence Safe
Absolute Maximum in Barge Impact Case	**68.3**	**t/m²**	<	469.61	Hence Safe
Absolute Minimum in Barge Impact Case	**67.7**	**t/m²**	>	0.00	Hence Safe
Absolute Maximum in Quasi Permanent	**73.6**	**t/m²**	<	469.61	Hence Safe
Absolute Minimum in Quasi Permanent	**73.6**	**t/m²**	>	0.00	Hence Safe

5. BEARING CAPACITY FOR SOIL (AS PER IS: 6403):

$$qd = cN_c s_c d_c i_c + q(Nq-1)s_q d_q i_q + .5B_\gamma N_\gamma s_\gamma d_\gamma i_\gamma W$$

Dia of well (m)	=	6	
Depth of foundation (D_f) m	=	19	(MSL-FND)
Depth of water Table (Dw) m	=	0	*from ground level.*
Cohesion (c) kN/m²	=	0	
Angle of Resistance (φ)	=	33.00	
Surcharge (q) (kN/m²)	=	171	

(γ_1) bulk kN/m³ = 20

Net Ultimate Bearing Capacity = ***9526.23*** kN/m²

	Shape Factor			Depth Factor			Inclination Factor		
Type	Sc	sq	s_γ	dc	dq	d_γ	i	iq	i_γ
Circular	1.300	1.200	0.600	2.17	1.58	1.58	1.00	1.00	1.00

Bearing capacity factor			
Nc	Nq	Nγ	Nφ
39.73	27.34	37.78	3.39

Summary for Net Bearing Capacity:

DIAMETER OF WELL	(φ)	Net BC (t/m²)	
6.00	33.00	971.07	Action comb. 1, Set-
6.00	27.45	469.61	Action comb. 2, Set-2
6.00	27.45	469.61	Seismic and Accidental Case

Angle of Resistance (φ) = *33.00* Set-1 *From Limit state code*

Angle of Resistance (φ) = *27.45* Set-2 *From Limit state code*

6. COMPARISION AND SUMMARY OF FOS OF WSM AND LSM

Working Stress Method

Load Combination	Allowable Moment tm	Allowable Resistance tm	FOS
LC1 (Wind)	24865	49730	2
LC2 (Seismic), LC3	39325	62920	1.6

Limit State Method

Load Combination	Allowable Moment tm	Allowable Resistance tm	FOS
LC1, LC2 (Wind, combination 1)	48890	49730	1.02
LC3, LC4 (Wind, combination 2)	24505	49730	2.03
LC5 (Seismic)	37908	62920	1.66
LC6 (Accidental)	29961	49730	1.66

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the Code/Guidelines/Manual, etc. from the date specified therein)

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